Geotechnical Report

Proposed Residential Development
Russell Road and Upper Ōrewa Road, Wainui

RILEY

Geotechnical Report
Proposed Residential Development
Russell Road and Upper Ōrewa Road, Wainui

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1.0 Introduction

The following report outlines a geotechnical engineering assessment for the proposed subdivision known as Delmore. The project involves the subdivision of just over 109ha in six (6) contiguous lots (88, 130, and 132 Upper Ōrewa Ro[redacted]A, 53B, and 55 Russell Road) and construction of a master-planned urban, residential development of approximately 1,250 dwellings.

This report has been prepared by Riley Consultants Ltd (Riley) to support the consenting process for the proposed Delmore Development. It presents the results of sub-surface investigations, in-situ and laboratory soils testing a[redacted]analysis together with our comments and recommendations pertaining to the satisfactory development of the site. It is intended to be used in support of a substantive applicatio[redacted]t-track approvals process.

2.0 Project Background

2.1 Site Description

The site is located on the northern s[redacted]oad and Upper Ōrewa Road, Wainui, comprising six rural properties including 53A Russell Road (Lot 1 DP 497022), 53B Russell Road (Lot 2 DP 497022), 55 Russell Road (Lot 1 DP 336616) and 88 Upper Ōrewa Road (Lot 2 DP 418770), 130 Upper Ōrewa Road (Lot 2 DP 153477) and 132 Upper Ōrewa Road (Lot 1 DP 153477). There is a paper road running north-south along the boundaries of 53B Russell Road and 88 Upper Ōrewa Road.

The site is bounded by neighbouring residential properties to the south and south-east, the Ara Hills residential development (currently under construction) to the north and east, and the Nukumea Scenic Reserve and other Significant Ecological Areas (SEA) to the north and west.

The site currently consists of lifestyle block/farm properties, comprising several associated residential dwellings, and an inactive deer farm. There are hay stores, cattle yards, farm sheds and shipping containers across the site. The vegetation comprises a combination of pasture, pine forest, native bush and windbreaks. The pine forest is in the north-eastern corner of the site while the bulk of the native bush is in the western and northern portions and generally confined within the alignment of the gullies. The pasture is generally located within the central and southern portions where there are 15m to 20m long windbreaks planted along paddock boundaries.

The site is characterised by a stream, which flows to the east through the middle of the site at 132 and 130 Upper Ōrewa Road, through the southern part of the site at 88 Upper Ōrewa Road and 53B and 53A Russell Road and then again through the middle of the site at 55 Russell Road.

All overland flow paths on-site flow down to this stream. The Auckland Council GeoMaps indicate an area of approximately 50m either side of the central stream, and smaller corridors around the tributary streams, are prone to flooding.

The topography of the site is characterised by gullies and slopes. In the eastern and central part of the site the moderate slopes, typically from 10° to 25° and steepening up to 27° in some places, grade from the top of the ridges and down into the gullies. At the eastern boundary, the maximum elevation difference from the top of the ridge down to the gully is approximately 45m at an approximate 20° gradient. The middle part of this area is shallower, with typical gradients between 5° and 15°, indicating a difference in underlying ground conditions. The western part of the site has more undulating and extreme topography, with steeper slopes, typically from 25° to 35°, steepening up to 40° to 4[redacted], particularly around areas of existing instability. The elevation difference from the hig[redacted] in the northern part of the site down to the gully is approximately 65m.

There are multiple small, dammed farm ponds on the site that appear to be shallow (~2m-3m) depth:

- Southern portion of 130 Upper Ōrewa Road – ~300m²
- Central northern portion of 130 [redacted]oad – ~1500m²
- North-eastern portion of 53B Russell Road – ~100m²
- Western portion of 53B Russel[redacted]0m²

The central constructed pond with[redacted]rewa Road is damming watercourses flowing to the south and has formed behind [redacted]ng a wetland to form with dense reed growth.

To the west of the northern constructed pond is an old farm structure once used for deer farming. To the east of the pond on the shallow slopes is a waste pit, covering an area of approximately 85m². There are other small storage sheds around site but most of the remaining farm structures are in the vicinity of the main dwelling on the property.

There are electrical and communication overhead lines, as well as other underground services, servicing the existing dwellings and properties on-site.

2.2 Proposed Development

The proposed development is understood to comprise approximately 1,250 residential lots and dwellings, neighbourhood parks, together with supporting transport and servicing infrastructure.

Subdivision and construction will occur in two stages, comprising six substages. Stage 1 comprises 53A, 53B and 55 Russell Road, and Stage 2 comprises properties 88, 130, and 132 Upper Ōrewa Road.

Preparatory earthworks across the site comprises cut of approximately 1,272,000m^3 and fill of approximately 953,000m^3. Earthworks plans indicate that cut and fill earthworks will take place over much of the site and be up to 16m depth. These earthworks are proposed to re-contour the site primarily to form the residential lots.

The designated two lane urban arterial road, running from SH1 and Grand Drive in the east along the site's northern side, and then down its western side to the southern boundary of the site, will be constructed as part of the project. There will be walking and cycling infrastructure along the side of this road. Homes within the site will be serviced by 27 local roads. The site's internal road network will connect to the external road network at 3 points. A total of 40 jointly owned access lots are used to connect the internal lots.

A total of 64 different housing typolo[redacted] used across the site. These include stand-alone and duplex housing options. The ground floor areas of the houses range between 97m^2 to 175m^2. Each housing typology will be pair[redacted]gy-specific landscaping comprising mostly native species, with a combination of low, medium, and high-level vegetation.

Walkways will be provided througho[redacted]ome routes provided from the site to the Scenic Reserve to the north. A neighbourhood park is shown indicatively within the middle of the site. Existing riparian native vegetatio[redacted]d and further enhancement planting will be undertaken. Existing areas of vegeta[redacted] consent notices will also be restored and enhanced with planting in places. These green spaces will be supported by on-street planting.

On-site effluent treatment will be [redacted]y a temporary treatment plant located in the southern portion of Stage 1. Treate[redacted]oposed to be disposed via specifically designed trenches located near the gully inv[redacted] of the treatment plant. The treatment plant and disposal trenches are being designed by Apex Water Ltd.

2.3 Site Geology

From a review of the 1:250,000 GNS Online Geological Map, the site is underlain by the following geological units:

- Northland Allochthon (Hukerenui Mudstone) – underlying the central/eastern portions of the site (central part of 53B Russell Road).
- East Coast Bays Formations – underlying most of the site.
- Pakiri Formation – underlying the northern part of the site.

The Waitemata Group deposits, represented as East Coast Bays Formation (ECBF) and Pakiri Formation (PF) materials, are sedimentary materials. The most widespread geological unit is the Miocene-age Waitemata Group that underlies the materials of the Northland Allochthon (NA) where present. The ECBF is described as comprising alternating sandstone and mudstone with variable volcanic content and interbedded volcaniclastic grit. The regional dip of the ECBF within the site is inferred to be 30° to the north-west. The PF comprises alternating thick bedded, volcanic rich, graded sandstone, and siltstone.

The materials of the Northland Allochthon are older materials that have been thrust over the younger ECBF and PF materials. The NA materials, mapped as Hukerenui Mudstone. These materials are typically described as sheared mudstone and are often red, green and grey in colour.

Tauranga Group Alluvium is also mapped as being present to the immediate south of the site. Based on-site stratigraphy and our experience with neighbouring sites, we consider that alluvial materials are likely to be present in the vicinity of waterways and gully inverts within the lower lying parts of the site. Tauranga Group generally comprises silts and sands, with the potential for localised peat lenses. These materials have generally been subjected to pre-consolidation; however, they may contain localised areas of very soft ground.

The approximate extent of the geolo[redacted] is shown on the appended Riley Sketch SK131 in Appendix H.

2.4 Aerial Photographs and GIS Records

A review of historical aerial photog[redacted]available on Auckland Council (Council) GIS Maps and Retrolens Historical Image Resource was undertaken. Based on a desktop review of the photographs, no obvious signs[redacted]d global type movement were noted in the historical aerial photographs. Howeve[redacted]bility, likely confined to surficial soils similar to those observed on-site as a part of our assessment were evident. Detailed descriptions of such instabilities associated with moderate to steep sloping areas and gully features are outlined in the geomorphology section below.

2.5 Geomorphology

Geomorphological mapping was undertaken by Riley on 8 April 2024 for Stage 1 of the site and 1 November 2024 for Stage 2.

2.5.1 Stage 1

The site is situated within an approximately east-west trending valley that drains to the east, with Russell Road following a ridgeline to the south that forms a drainage divide. The topography is dominated by a number of generally north-south trending ridges and gullies that grade down to a stream at the base of the valley near the central portion of the site that flows toward the coast. There are tributary valleys and streams across the site that are to remain as part of the development. The site consists of three main tributary catchments, and approximately three minor catchments.

The more elevated areas of the north-eastern slopes consist of benches with gently inclined terraces above relatively short steeper slopes. The southern north facing slopes show signs of localised shallow instability, predominantly within the upper reaches of gully features.

There were areas of localised instability across the site. Areas of more significant instability were identified on the steeper slopes located to the north of 53B Russell Road, and near the centre of the 55 Russell Road to the east of the property boundary fence, north of the central stream.

The upper reaches of the southern trending gully at 53B Russell Road is dominated by hummocky undulating ground, with mid slope benches and swampy waterlogged ground with dense pockets of reeds. These characteristics are typically associated with shallow soil movement and poor drainage. This is an area inferred to be underlain by materials of the Northland Allochthon and is in the vicinity of the contact between geological units.

Within 55 Russell Road and to the east of the boundary fence with 53B, a notable more prominent relic slip is evident. This was observed as a broad scarp feature running perpendicular to the slope direction, approximately 100m in length. This runout from this shallow slump movement extends southward toward the central stream, and is characterised by hummocky, undulating and waterlogged ground. The runout has been incised over time, forming channels through the weaker unconsolidated material.

No obvious spring locations were noted during the geomorphological walkover on-site.

2.5.2 Stage 2

South

Stage 2 is located to the west of Stage 1 and extends around the same east-west trending valley feature with the same main w[redacted]ding stream flowing through the site. The north-western portion of Stage 2 forms the head of a larger tributary catchment on the site, with a number of secondary tributary streams flowing south.

Along the southern boundary of Stage 2 a broad spur stems off from an approximately north-west trending ridgeline. U[redacted]oad generally follows the ridgeline along the southern boundary of the site. A[redacted]elling is situated centrally on this spur feature. The spur generally tapers out at moderate to steep slopes towards the west, north and east.

To the north and west of the spur, slopes generally grade at moderate to steep slopes into the stream of the east-west channel, to the north, and into a northwest orientated gully to the west. Dense vegetation is present within the gullies and adjacent the east-west orientated central stream. On the lower reaches of the northern slopes of the spur is a minor constructed pond for stock with an overflow to the north that leads into the central stream. Localised shallow instability was observed to the east of the spur, predominantly within the upper reaches of the gully feature.

There were noted areas of localised instability on the steeper slopes located to the north of the spur, the south facing slopes on the northern side of the central stream, and on the eastern side of the spur feature. The north facing steep slopes of the spur show evidence of soil creep as evidenced by small terrace features parallel to the slope and hummocky ground. There is a mid-slope bench that is waterlogged with pockets of reeds. The southern facing slopes on the northern side of the central stream show signs of large historic instability with a series of benches being formed. There is evidence of soil creep across adjacent slopes in the form of undulating and hummocky ground, terrace features, and trees tilted/rotated back in the direction of the slope.

In the eastern part of the southern site, there are two incised gullies sloping west into the vegetated central stream which show evidence of potential tunnel gully erosion. The presence of tunnel gulley erosion is assessed to be attributed to the underlying ground conditions, likely more silty and sandy materials.

North

The east-west trending stream passes the toe of the spur within the southern portion of Stage 2 and extends to the western site boundary. It also branches to the north into the northern portion of Stage 2.

The northern portion of Stage 2 is dominated by a north-south facing valley basin. The western boundary consists of an approxima[redacted]-south orientated ridge, with a steep backdrop towards the west and more moderate to gentle runout towards the east towards the central stream. A prominent spur branches off the rid[redacted]south-east toward the centre of the site.

The northern boundary consists of a south-west-north-east orientated ridgeline with multiple approximately south trending spurs[redacted]ullies extending into the site. These are moderately to steeply inclined slopes dominated by the growth of gorse and other vegetation.

The eastern boundary of Stage 2 c[redacted]dge feature extending in a south-easterly direction.

The slopes tend to shallow towards th[redacted]f this portion of Stage 2 down to a basin. The flow paths here converge into a wetland that discharges to the south. The lower reaches of the basin were noted to consist of boggy, wa[redacted]nd, with dense localised pockets of reeds. To the south there is a culvert crossing th[redacted]e culvert has been undermined on the southern side, where a small slip has occurred. To the south the land becomes densely vegetated adjacent the stream.

There were noted areas of instability on the steeper western and east facing slopes either side of the central stream. The steeper slopes show signs of shallow soil movement in the form of small terraces on more moderate gradients and shallow localised slumping on steeper slopes and near gully heads. Localised instability was noted around the edges of streams, especially those that have been relatively deeply incised.

2.6 Related Reports

The following geotechnical reports that are available to us have been reviewed during the preparation of this assessment:

- Geotechnical Assessment Report, prepared by CMW Geosciences Ltd for Ara Hills Stage 3A (formerly Stage 8), dated 15 June2021, ref: AKL2020-0312AB Rev.0;
- Geotechnical Design Report – South Eastern Package, prepared by Tetra Tech Coffey for Ara Hills Stage 2, dated 8 November 2021, ref: AKLGE290955AA-AA;
- Geotechnical Design Report – Western Package, prepared by Tetra Tech Coffey for Ara Hills Stage 2, dated 26 November 2021, ref: AKLGE290955AA-AB; and

- Geotechnical Completion Report, prepared by CMW Geosciences Ltd for Ara Hills Stage 3-A1, dated 15 September 2022, ref: AKL2020-0312AI Rev.2.

3.0 Investigations

3.1 Fieldwork

Riley has undertaken two phases of site investigation across Stage 1 and Stage 2 in November and December 2024, comprising test pits, hand auger boreholes, and machine boreholes. The scope of the completed investigation is summarised below.

- Phase 1
    - 20 hand auger boreholes [redacted] HA11 to HA22) to maximum 5m depth, or refusal.
        - Shear vane tes[redacted]ly undertaken at 0.5m intervals.
        - Scala penetrometer testing (Scala) was carried out in place of shear vanes in granular ma[redacted]
        - Scalas were carried out at the base of hand auger boreholes to maximum 2m depth, or ref[redacted]
        - 11 groundwater [redacted]ndpipes (HA1, HA2, HA5, HA6, HA7, HA11, HA13, HA16, HA19, HA20 and HA21) were installed, with a response zone from base of hand auger [redacted] ground level.
    - 53 test pits (TP1 to TP10, TP12 to TP54) to maximum 6.0m depth.
        - Shear vane t[redacted]ally undertaken at 1.0m intervals.
- Representative soil samples were recovered from selected test pit locations for subsequent laboratory testing.
- Phase 2
    - 65 hand auger boreholes (HA101 to HA165) to maximum 5m depth, or refusal.
        - Shear vane tests were typically undertaken at 0.5m intervals.
        - Scalas were carried out at the base of selected hand auger boreholes to maximum 2m depth, or refusal.
    - Six machine boreholes (MH01 to MH06) to maximum 19.5m depth.
        - Eight groundwater monitoring standpipes installed as nested piezometers (two per borehole) in four machine boreholes (MH01, MH02, MH04, and MH06).

The locations of the site investigation points are shown on sketch, site plans Sketches: 240065-SK110 to SK124 (Appendix H). The results of all in-situ testing, together with descriptions and depths of strata encountered during the drilling are presented on the test pit and borehole logs appended in Appendix A.

3.2 Laboratory Testing

The following laboratory tests were scheduled on collected samples in Stages 1 and 2. The tests were undertaken by Babbage Geotechnical Laboratory, which is an IANZ Accredited Testing Laboratory, in accordance with NZS4402. The results of these laboratory tests are discussed in Section 4.7 and appended in Appendix D.

- Eight Standard Compaction tests (standard five point with shear vane tests).
- Six Atterberg Limit tests.
- Five Particle Size Distribution Test by Hydrometer.
- Two Soaked CBR Tests

The following tests were undertaken by Riley. Riley is not an accredited laboratory.

- 36 water content tests.

3.2.1 Water Content Testing

Thirty-six samples were obtained for [redacted]ent testing. These tests were carried out by Riley in accordance with NZS4402 [redacted] 2.1. The test results are appended and summarised in Section 4.7. This testing was carried out to assist with establishing soil moisture conditioning requirements for potential earthworks.

3.2.2 Standard Compaction Testing

Eight Standard Compaction tests t[redacted]t Number 4.1.1 were carried out on representative samples of ECBF and NA. This testing was carried out on potential cut/borrow material to establish appropriate compaction control criteria. The results are outlined in Section 4.7 and the laboratory reports are appended.

3.2.3 Atterberg Limit Testing

Six Atterberg Limits tests were carried out in accordance with NZS4402 Test Numbers: 2.1, 2.1, 2.3, and 2.4 on representative samples of ECBF and NA. The results are outlined in Section 4.7.

3.2.4 Particle Size Distribution Testing

Five particle size distribution tests were carried in accordance with NZS4402 Test Number 2.8.4 on samples of ECBF. Test results are appended and are outlined in Section 4.7.

3.2.5 Soaked CBR Testing

Two soaked CBR tests were carried out in accordance with NZS4402 Test Numbers 2.1, 4.1.1 and 6.1.1. Test results are appended and are outlined in Section 4.7.

4.0 Investigation Findings

The finding of the investigation outlined in Section 3.1 are summarised below.

4.1 Topsoil

Topsoil was encountered within all test pits and hand augers, up to a depth of approximately 0.4m below ground level (bgl). TP51 (located in the northwestern part of Stage 2) encountered a 100mm band of peat underlying the topsoil layer.

4.2 Alluvium

Alluvial soils were encountered in HA10[redacted]118, and HA135 (which were drilled within the gullies), to depths between `1.5m and 4.7m depth. The alluvial soils typically comprised clayey silt and silty clay, ranging from moist to wet and [redacted]lets and organic material, including wood, flax and humus. HA135 also recorded a layer of silty sand overlying the residual weathered soils.

4.3 Colluvium [redacted]

Colluvium was encountered within nin[redacted]4-7, 28, 42, 47-48, 54) and eight hand augers (HA2, 6, 7, 106, 110, 122-124), typically be[redacted] and 0.9m bgl underlying topsoil. These soils generally consisted of stiff to very stiff silts and some clays.

Notable depths of Colluvium were e[redacted] in HA122, HA123 and HA124 in the central part of Stage 1, specifically to depths of 3.0m, 2.35m, and 2.1m, respectively (3.0m due to target depth of hand auger), and in TP47, TP48, a[redacted] central eastern part of Stage 2, specifically to depths of 3.5m, 5.0m, and 4.9m, re[redacted]m due to target depth of test pits). It is inferred that these thick colluvium deposits are associated with previous areas of instability.

Colluvium within these three test pits (TP47, TP48, and TP54) typically consisted of firm to stiff silts with some clays. In TP48 and TP54, layers of medium to coarse, sand and gravel between 3m and 5m bgl were noted.

4.4 East Coast Bays Formation

ECBF residual soils were encountered in the majority of test pits underlying the topsoil and colluvium and generally comprise stiff to hard silts, clays, and sands. The depth of residual soils varied from 2.8m to greater than 5m depth across the site.

It is difficult to distinguish between coarser ECBF residual soils from the Pakiri Formation. Although PF is identified on the geological map as underlying the northernmost part of the site PF soils were not recorded during the site investigation. The engineering properties of the residual soils are similar and would not change the recommendations made in this report.

In 29 test pits across the site, weathered ECBF rock was encountered underlying the residual ECBF soil. The encountered ECBF rock was typically moderately weathered extremely weak to weak siltstone and sandstone.

Scala testing at the base of hand augers within ECBF typically reached refusal at depths between 4.3m to 6.9m bgl. Refusal of the Scala was not reached within the base of three hand augers only (HA19, 20, and 22).

ECBF weathered rock was encountered in five machine borehole locations (MH01, MH02, MH03, MH04 and MH06) underlying varying thicknesses of residual soils (up to 9.35m in MH02) and the Northland Allochthon in MH01, at 7.65m depth. It was typically logged as highly weathered to unweathered grey siltstone and sandstone, with bedding planes ranging from 0° to 10° and joints, typically smooth, planar and with no infill.

4.5 Northland Allochthon - Hukerenui Mudstone

The residual soil of the Hukerenui M[redacted]f the Northland Allochthon Group materials was encountered underlying the topsoil and Colluvium within 7 test pits (TP1, 5-7, 19-20, 23) on Stage 1 of the site. The Northland Allochthon was en[redacted]e central area of 53B, as well as at the western boundary. Both areas are associated [redacted] wet ground, and shallower slope gradient.

The residual soils generally comprise[redacted]ds and were generally dry to moist, above the groundwater table. In all these test pit locations, the residual soil was underlain by weathered rock, typically encountered between 2[redacted] bgl. The Hukerenui Mudstone was typically described as completely to moder[redacted]d extremely weak to weak siltstones and sandstones. The siltstone was typically sheared and muddy.

Cores of slightly weathered Hukere[redacted]one were recovered in MH01 and MH05. MH05 reached target depth at 9.1m, recovering slightly weathered dark grey sandstone. MH01 recovered slightly weathered light grey and [redacted]ly sheared mudstone to 7.65m depth where it reached the unconformable conta[redacted]st Coast Bays Formations.

4.6 Groundwater Level and Monitoring

Groundwater was encountered in multiple investigation locations during the investigation completed in November and December 2024.

Within the test pits, groundwater was encountered during excavation within 12 of the 53 locations (TP15, 17, 21, 23,2 6-27, 37-38, 41, 43, 48, 51) at depths between 2.0m and 5.2m below existing ground level.

During drilling of the hand augers, groundwater was encountered in 13 of the 20 locations (HA1-3, 5–7, 11–13, 15–17, 19) at depths between 1.5m and 4.8m below existing ground level. Groundwater monitoring standpipes were installed within 11 hand augers (HA1, 2, 4-7, 11, 13, 16, 19-21), as detailed in Table 1. The standpipes on Stage 1 (HA1, 2, 4-7, 11) were installed between 6-7 November 2024 and the standpipes on Stage 2 (HA13, 16, 19-21) were installed between 18-20 November 2024.

Groundwater monitoring standpipes were installed as nested piezometers in four of the machine boreholes (MH01, MH02, MH04, and MH06) to capture potential shallow and deep groundwater regimes.

Groundwater monitoring of the installed standpipes was initially undertaken for two consecutive weeks following installation and then will be monitored at monthly intervals.

Standpipes in HA13 and HA16 are located in areas of reeds. Monitoring of these points indicates an elevated groundwater level in comparison to other monitoring points. Groundwater levels within such areas are often elevated all year round. HA19 is located within a hummocky area with a head scarp upslope. The monitoring of this locations has also shown elevated groundwater levels.

Table 1: Measured Groundwater Levels in Piezometers

Stage	Hole ID	Response Zone (m bgl)	Ground-water Level[1] (m bgl)	Groundwater Levels (m bgl)				
				21/11/24	28/11/24	05/12/24	19/12/24	16/01/25
Stage 1	HA1	1.0 – 5.0	[redacted]	[redacted]	No access[2]	-[3]	No access[2]	No access[2]
	HA2	1.0 – 5.0	3.0	2.53	2.59	-	2.96	2.62
	HA5	1.0 – 5.0	[redacted]	[redacted]	4.77	-	N/E[4]	N/E
	HA6	1.0 – 4.5	4.37	3.69	3.69	-	3.48	3.87
	HA7	1.0 – 5.0	[redacted]	[redacted]	No access[2]	-	3.06	3.74
	HA11	1.0 – 4.1	3.3	3.29	3.35	-	3.19	3.46
	MH01	1.5 – 2.5	[redacted]				0.69	1.26
	MH01	4.5 – 6.5	-				2.91	2.39
	MH02	2.0 – 6.0	[redacted]				3.28	3.93
	MH02	8.0 – 11.0	-				3.58	3.93
	MH04	6.0 – 8.0	-				5.49	5.84
	MH04	13.0 – 15.0	-				9.30	9.42
	MH06	3.0 – 5.0	-				3.04	4.15
	MH06	6.5 – 8.5	-				6.45	6.92
Stage 2	HA13	1.0 – 4.0	2.26		1.98	2.07	2.07	2.66
	HA16	1.0 – 4.7	2.6		2.26	2.33	2.22	2.73
	HA19	1.0 – 5.0	2.96		0.81	0.94	Dest-royed[5]	-
	HA20	1.0 – 4.0	N/E		N/E	N/E	N/E	N/E
	HA21	1.0 – 5.0	N/E		4.83	4.92	N/E	N/E

[1] Groundwater level measured during drilling of hand auger and machine borehole.

[2] Monitoring points were unable to be accessed due to cattle.

[3] Dashed line denotes not measured.

[4] N/E denotes not encountered.

[5] Destroyed by cattle.

4.7 Laboratory Test Results

4.7.1 Water Content

Riley carried out moisture content tests on a total of 36 samples. Results are presented in Table 2.

Table 2: Soil Water Content Percentages

Stage	Test Number	Geological Unit	Investigation ID	Depth (m)	Water Content (%)
Stage 1	1	Northland Allochthon [redacted]	TP7	0.6-0.8	49.7
	2		TP7	2.0-2.3	47.3
	3		TP7	2.8-3.1	36.6
	4		TP7	4.2-4.5	33.3
	5	[redacted] East Coast B[redacted] [redacted]	TP10	1.5-1.8	39.4
	6		TP10	2.4-2.7	38.5
	7		TP10	3.6-3.8	22.6
	8		TP16	1.0-1.2	52.3
	9		TP16	2.0-2.2	50.3
	10		TP3	1.0-2.0	43.9
	11		TP3	4.0-5.0	41.9
	12	Northlan[redacted]n	TP1	0.5-2.0	66.3
	13		TP1	2.9 -4.6	28.4
Stage 2	14	[redacted]	Not Tested		
	15				
	16	East Coast Bays Formation	TP41	1.0-2.0	36.5
	17		TP41	3.2-4.6	41.1
	18		TP27	1.1-2.1	53.3
	19		TP27	4.6-5.2	52.5
	20		TP27	5.2-5.4	35.4
	21		TP25	1.0-2.0	40.1
	22		TP25	4.0-5.5	32.0
	23		TP25	3.5-4.5	48.2
	24		TP30	0.5-1.2	48.2
	25		TP30	1.5-2.1	53.1
	26		TP30	2.8-3.1	52.7
	27		TP30	3.45-4.1	52.0
	28		TP36	1.1-2.0	55.2
	29		TP36	2.2-3.3	54.3
	30		TP36	3.8-4.6	41.7
	31		TP38	1.0-2.0	49.7
	32		TP39	0.3-0.9	40.9

Stage	Test Number	Geological Unit	Investigation ID	Depth (m)	Water Content (%)
	33		TP39	1.0-2.0	47.3
	34		TP39	3.5-4.5	58.9
	35		TP47	1.0-2.0	32.4
	36		TP47	4.0-5.0	30.9
	37		TP49	1.0-2.0	34.9
	38		TP49	4.0-5.0	53.1

The results indicate that there is a relatively wide spread of moisture content results. They also indicated the upper 2m-3m of the soil profile has moisture contents typically 10%-20% higher the deeper soils. The results are summa[redacted]ble 3.

Table 3: Summary of Riley Soil Moist[redacted]

Stage	Geological Unit	Maximum Moisture Content (%)	Minimum Moisture Content (%)	Average Moisture Content (%)
1	Northland Allochthon	66.3	28.4	43.6
1	East Coast Bays Formation	[redacted]	22.6	41.3

4.7.2 Standard Compaction

The Standard compaction test results[redacted]marised in Table 4. Test results are appended.

Table 4: Standard Compaction Te[redacted]

Stage	Investigation ID	Geological Unit	Depth (m)	As Received Moisture Content (%)	Maximum Dry Density (t/m³)	Optimum Moisture Content (%)
Stage 1	TP3	East Coast Bays Formation	4.0 - 5.0	42.4	1.54	20
	TP7	Northland Allochthon	0.6 - 0.8	49.4	1.25	36
	TP10	East Coast Bays Formation	2.4 - 2.7	36.1	1.53	21
	TP10		3.6 – 3.8	23.4	1.60	21
Stage 2	TP25	East Coast Bays Formation	4.0 - 5.0	33.5	1.52	21
	TP27		4.6 - 5.2	52.3	1.33	31
	TP41		3.6 - 4.6	48.1	1.43	25
	TP47		1.0 - 2.0	37.2	1.42	29

The standard compaction tests returned a maximum dry density of 1.25t/m³ (TP7) at an optimum moisture content (OMC) of 36% for a sample comprising Northland Allochthon soils. The OMC was approximately 13% lower than the as received soil moisture content.

For samples taken from ECBF soils the maximum dry densities were between 1.33t/m³ (TP27) and 1.60t/m³(TP10). The test results indicate that the OMC is approximately 15% to 20% below the natural moisture content.

As with the soil moisture contents presented in Tables 2 and 3 above, the as received soil moisture content test results indicate that there is significant variability in the soil moisture. The test results also indicate that soil moisture content conditioning in the order of 10% to 20% will likely be required to dry back to the optimum moisture content.

4.7.3 Atterberg Limits and Linear Shrinkage

Atterberg Limits testing was undertak[redacted]representative samples of materials, between 0.6m and 5.0m depth. Testing was undert[redacted] 4402:1986 and is IANZ endorsed. Test results are presented in the Table 5. Full laborat[redacted]pended. The testing was undertaken to assist with assessing the site soil plasticity c[redacted] characteristics. Further expansive soil testing will be required as part of geotechnical completion reporting following earthworks.

Table 5: Atterberg Limit and Linear S[redacted]ults

Stage	Investigation ID	Geological Unit	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	As Received Water Content (%)
Stage 1	TP3	East Coast Bay[redacted]	4.0 - 5.0	46	26	20	43.2
	TP7	Northland Allochthon	0.6 - 0.8	86	33	53	49.6
	TP10	East Coast [redacted]	3.6 - 3.8	52	18	34	24.0
	TP16		1.0 - 1.2	109	34	75	50.5
Stage 2	TP41	East Coast Bays Formation	3.2 - 4.6	55	23	32	47.5
	TP47		1.0 - 2.0	76	25	51	37.4

The Atterberg Limit test results indicate that all samples tested have a USCS classification of either CL (clay of low plasticity, TP3) or CH (clay of high plasticity). We do note however, that for the TP10 sample, only the soil fraction passing the 425µm sieve was used. While this is consistent with the testing standard, it could result in the plasticity being overstated for this sample. In general, field descriptions of the sampled material indicated a lower plasticity and clay content than those reported in the laboratory tests.

Soils that are dry of the plastic limit will not behave in a plastic manner and will likely become difficult to compact to an engineered standard. Considering the proximity of the OMC (see Section 4.7.2 above) to the plastic limit, it will be important that care is taken to ensure that soil moisture contents are not more than 2-3% below the OMC following conditioning.

4.7.4 Particle Size Distribution

The particle size distribution tests were carried out on samples obtained from TP10, TP30, TP36, and TP41 to assess the soil textures for assessment of the soil category for treated effluent soakage. Test results are presented in Table 6.

Table 6: Particle Size Distribution Test Results

Stage	Geological Unit	Investigation ID	Depth (m)	Clay (%)	Silt (%)	Sand (%)	Gravel (%)
Stage 1	East Coast Bays Formation	TP10	2.4 - 2.7	18	23	58	1
		TP10	3.6 - 3.8	24	63	13	0
Stage 2		TP3[redacted]	3.5 - 4.1	26	64	10	0
		TP36	3.8 - 4.6	12	25	63	0
		TP[redacted]	[redacted] 4.6	20	44	36	0

4.7.5 California Bearing Ratio (CBR) [redacted]

Two soaked CBR tests were undertaken on soil samples recovered from TP30 at 3.5m to 4.1m depth and TP36 at 3.8m to 4.6m depth. T[redacted]dertaken to assist with assessments of the preliminary subgrade CBR for road pavement design. A portion of the TP36 sample comprised completely weathered sandstone and siltstone which was observed by the laboratory to be easily broken over a 9.5mm sieve. The test[redacted] soaked CBRs of 1% and 2%, respectively. We note that the samples tested were described as being sandy with low to moderate plasticity and both had moisture contents above th[redacted] results are considered to be atypical for the geological units present here. Th[redacted]n of the sandy soil composition and elevated moisture content is a likely explanation for the low laboratory CBR values.

5.0 Geotechnical Considerations

Based on the results of our field investigations, Riley considers the proposed development should be suitable for the ground conditions encountered, subject to the recommendations below.

5.1 Slope Stability

Qualitative and quantitative stability assessments have been carried out for the purpose of addressing the requirements of Section E36 of the AUP together with relevant considerations with respect to Section 106 of the RMA.

5.1.1 Qualitative Slope Stability Assessment

The instability features described in Section 2.5 together with Northland Allochthon deposits and its interface with the ECBF present complex stability issues that will need to be addressed. Similar land instability features were understood to be present at the adjacent Ara Hills development.

As outlined in Section 2.5, areas of the development are affected by existing significant instability features by including head scarps, hummocky, undulating and waterlogged ground across portions of the site. Due to the presence of these instability features, we consider that such portions of the land will require stability enhancement to ensure suitable accessways and building platforms are available for future residential development. We consider that these enhancements will need to comprise a combination of palisade walls, shear keys, buttress fills, and mechanically stabilised earth fills.

However, there are also some localised areas (particularly in the south-east corner of the development) that are free of observed existing instability features and where gradients are sufficiently gentle such that they unlikely to require stability enhancement works to support future dwelling development.

[redacted]

5.1.2 Quantitative Slope Stability Assessment

To provide a quantitative assessment [redacted]le Factor of Safety (FoS) against instability for the post-development ground surface, analyses have been undertaken for cross sections A to P for Stage 1 and Q to AD for Stage 2. [redacted]

These analyses also include a cross-s[redacted]he post development ground surface in the vicinity of the proposed water/wastew[redacted]plant and potential effluent discharge trench locations. For the potential effluent discharge trench locations, groundwater conditions within the trenches have been assumed to be at the surface for all analysed conditions. This essentially means that through the modelling process[redacted]ough the groundwater table was lower during investigations in the vicinity) the ground downslope of the trenches has been treated as saturated for all analysed conditions. We con[redacted]s an appropriately conservative approach.

For the purpose of the stability assessments, we have utilised the software Slide 2. The analyses have been undertaken using the Morgenstern Price method of analysis for non-circular slip surfaces. The stability analysis results are outlined below. The analyses consider long term groundwater levels, temporary saturated ground conditions and a ULS seismic scenario. For the ULS seismic scenario a peak ground acceleration of 0.19g (as per the MBIE guidelines) was used.

5.1.3 Geotechnical Model and Analysis Parameters

With respect to the comments on site geomorphology in Section 2.5 above, deep-seated instability within the underlying rock mass is considered unlikely. The instability features that have been observed within the site are considered likely to have occurred as a result of saturation of the soil profile during extreme/seasonal wet periods or in the case of portions of Stage 2 due to saturation of a sandy horizon just above the underlying rock. Consistent with the comments on site geomorphology, we have considered circular and non-circular instability mechanisms within the soil profile.

The stability analysis considered assumed normal groundwater conditions, extreme saturated conditions and ULS seismic conditions. The normal groundwater level is inferred based on the groundwater encountered at the investigation locations. The extreme groundwater condition was modelled as 80% saturation of the surficial soil layers above the long-term water table using an R_u coefficient of 0.44 for most cross-sections.

Extreme (transient) groundwater conditions are assumed to occur infrequently and persist for a relatively short period following heavy rainfall events. This is approximately equivalent to a 1m deep water table during the transient scenario for locations where the existing groundwater level is at 5m depth. Where the existing water table is shallower, the approximately equivalent water table would be closer to the ground surface. We consider this approach is appropriately conservative.

The normal groundwater level was used for the ULS seismic event case. For all cases, because there will be subsoil drains beneath the subdivisional fill, it was analysed as being drained.

A FoS greater than 1.5 is required for permanent slopes under long-term groundwater conditions, while a FoS greater than 1.3 is acceptable for temporary transient groundwater conditions during temporary elevated conditions. A FoS [redacted]han 1 is required for the ULS seismic event scenario. These target FoS are generally consistent with the Auckland Council Code of Practice for Land Development, with the exception [redacted] case which the proposed target is more conservative.

In accordance with the Ministry o[redacted]ation and Employment (MBIE) and the New Zealand Geotechnical Society (NZGS) Module 1 Guidelines, a ULS Peak Ground Acceleration (PGA) of 0.19g has been used for the [redacted]on. This value has been determined with a ULS (Ultimate Limit State) return perio[redacted]

For these analyses, the effective stres[redacted]meters presented in Table 7 were adopted.

Table 7: Soil Parameters

Material Names	Unit Weight (kN/m³)	Cohesion (kPa)	Phi (deg)
Engineered Fill	18	8	32
Gully mullock	17	2	22
Colluvium	17	2	25
Firm NA	18	3	22
Stiff NA	18	5	25
Very Stiff NA	18	5	30
Weak NA layer	19	0	12
Very weak NA rock	20	5	25
Weak NA rock	20	5	30
Medium Dense ECBF	18	2	30
Firm ECBF	18	5	26
Stiff ECBF	18	5	28
Very Stiff ECBF	18	7	30
Very weak ECBF rock	20	10	35
Weak ECBF rock	20	20	40

Full details of the stability analyses are appended and summarised below.

5.1.4 Quantitative Stability Analysis Results

The results are summarised in Table 8 and Table 9 below.

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

Table 8: Summary of Stability Analysis Results – Post Development (Stage 1)

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
A	Long Term Groundwater Conditions	1.5	<1	FoS less than the required values for all cases. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	<1	
	Seismic	1.0	<1	
	Long Term Groundwater Conditions (Stability enhancement)	1.5	1.16	Adequate FoS achieved for slip surfaces encroaching into the lots (i.e. FoS less than the target values occur outside the lots), with inclusion of stability enhancement measures for long term and temporary elevated groundwater conditions. FoS less than 1 for seismic conditions but deformations calculated using the Jibson (2007) method are between 0mm and 5mm, which is considered to be acceptable based on MBIE guidelines. Proposed Enhancement: Tiered retaining walls between the road and the northern boundary (1.5m spacing, 300kN shear capacity per pile, approx. 8m to 10m long). Palisade walls (1.5m spacing, 200kN shear capacity per pile, piles approx. 5m long) required at the base of the central and upper fill areas. The lower fill area needs to be substantially undercut down to the NA rock (3m-5m depth over a width of 40m-50m). Geogrid MSE slopes (20kN/m long-term strength at 0.6m vertical intervals, approx. 3m-5m long) required for fill batters.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	[redacted]	<1	
	Seismic (Stability enhancement)	[redacted]	<1	

[redacted]

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
B	Long Term Groundwater Conditions	1.5	1.20	FoS less than the required values for long term and temporary elevated groundwater conditions. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	<1	
	Seismic	1.0	<1	
	Long Term Groundwater Conditions (Stability enhancement)	1.5	1.99	Adequate FoS achieved with inclusion of stability enhancement measures. Option 1: Palisade wall (1.5m spacing, 280kN shear capacity per pile, piles approx. 16m long) with geogrid MSE slope (20kN/m long term strength at 0.6m vertical intervals, approx. 8m long) above over lower portions of the slope. Soil nails (1.2m spacing, 50kN tensile capacity, 7m long) over the upper portion. Option 2: As for option 1 but soils nails switched for a 1v:3h slope and a 3m high cantilever retaining wall (1.5m spacing, 120kN shear capacity per pile).
	Temporary Elevated Groundwater Conditions	[redacted]	1.36	
	Seismic (Stability enhancement)	[redacted]	1.36	
C	Long Term Groundwater Conditions	1.5 [redacted]	1.2	FoS less than the required values for all cases. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3 [redacted]	<1	
	Seismic	[redacted]	<1	
	Long Term Groundwater Conditions (Stability enhancement)	[redacted]	1.75	Adequate FoS achieved with inclusion of stability enhancement measures. Proposed Enhancement: Geogrid MSE slope (20kN/m long term strength at 0.6m vertical intervals, approx. 3m long) over the lower portions of the slope. Colluvium to be excavated and replaced with engineered fill. Soil nails (1.2m spacing, 50kN tensile capacity, 8m long) over the upper portion of the slope.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	[redacted]	1.43	
	Seismic (Stability enhancement)	1.0	1.28	

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
D	Long Term Groundwater Conditions	1.5	1.66	FoS less than the required values for long term and temporary elevated groundwater conditions. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	1.21	
	Seismic	1.0	1.27	
	Long Term Groundwater Conditions (Stability enhancement)	1.5	1.86	Adequate FoS achieved with inclusion of stability enhancement measures. Proposed Enhancement: Geogrid MSE slope (20kN/m long term strength at 0.6m vertical intervals, approx. 4m long) above over lower portions of the slope. Retaining walls required between platform benches and around the pond where localised steep gradients are present.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	[redacted]	1.38	
	Seismic (Stability enhancement)	[redacted]	1.41	
E	Long Term Groundwater Conditions	[redacted]	1.17	FoS are adequate for the northern slope. For the southern slope they are less than the required values for long term and temporary elevated groundwater conditions. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	[redacted]	<1	
	Seismic	1.0	<1	
	Long Term Groundwater Conditions (Stability enhancement)	[redacted]	>2	Adequate FoS achieved for the southern slope with inclusion of stability enhancement measures. Proposed Enhancement: Palisade wall (1.5m spacing, 120kN shear capacity per pile, piles approx. 7m long) with geogrid MSE slope (20kN/m long term strength at 0.6m vertical intervals, approx. 4m long) above over lower portions of the slope.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	1.3	1.92	
	Seismic (Stability enhancement)	1.0	1.38	

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
F	Long Term Groundwater Conditions	1.5	1.75	FoS are adequate for all cases except the temporary elevated groundwater conditions where slip surfaces below the target value occur on the upper southern slope adjacent to the property boundary.
	Temporary Elevated Groundwater Conditions	1.3	1.34	
	Seismic	1.0	1.26	
	Long Term Groundwater Conditions (Stability enhancement)	1.5	>2	Adequate FoS achieved for the southern slope with inclusion of stability enhancement measures. Proposed Enhancement: Approx 2.5m high cantilever retaining wall (1.5m spacing, 120kN shear capacity per pile, piles approx. 5m long) with the slope gradient above adjusted to 1v:3h.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	[redacted]	1.71	
	Seismic (Stability enhancement)	[redacted]	1.24	
G	Long Term Groundwater Conditions	[redacted]	>2	Adequate FoS are available for all cases considered. Stability enhancement measures are not required.
	Temporary Elevated Groundwater Conditions	[redacted]	>2	
	Seismic	1.0	>2	
H	Long Term Groundwater Conditions	[redacted]	1.62	FoS less than the required values for temporary elevated groundwater conditions. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	[redacted]	1.05	
	Seismic	1.0	1.12	
	Long Term Groundwater Conditions (Stability enhancement)	1.5	1.99	Adequate FoS achieved with inclusion of stability enhancement measures.

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
	Temporary Elevated Groundwater Conditions (Stability enhancement)	1.3	1.33	Proposed Enhancement: Palisade wall (1.5m spacing, 120kN shear capacity per pile, piles approx. 3m long) with geogrid MSE slope (20kN/m long term strength at 0.6m vertical intervals, approx. 8.5m long with at least 3 layers 12.5m long).
	Seismic (Stability enhancement)	1.0	1.32	
I	Long Term Groundwater Conditions	1.5	1.31	FoS less than the required values for all cases. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	1.15	
	Seismic	[redacted]	<1	
	Long Term Groundwater Conditions (Stability enhancement)	[redacted]	1.59	Adequate FoS achieved with inclusion of stability enhancement measures for long term and temporary elevated groundwater conditions. FoS less than 1 for seismic conditions but deformations calculated using the Jibson (2007) method are between 0 and 3mm, which is considered to be acceptable, based on MBIE guidelines. Proposed Enhancement: Palisade wall (1.5m spacing, 280kN shear capacity per pile, piles approx. 8m long) with geogrid MSE slope (20kN/m long-term strength at 0.6m vertical intervals, approx. 8m long).
	Temporary Elevated Groundwater Conditions (Stability enhancement)	[redacted]	1.41	
	Seismic (Stability enhancement)	[redacted] 1.0	<1	

[redacted]

[redacted]

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
J	Long Term Groundwater Conditions	1.5	<1	FoS less than the required values for all cases. Stability enhancement measures required. Groundwater at the surface in the gully for all cases.
	Temporary Elevated Groundwater Conditions	1.3	<1	
	Seismic	1.0	<1	
	Long Term Groundwater Conditions (Stability enhancement)	1.5	1.55	Adequate FoS achieved with inclusion of stability enhancement measures. The minimum FoS for the seismic case occurs outside the embankment. FoS for slip surfaces encroaching within the fill embankment is >1, which is acceptable. Proposed Enhancement: Geogrid MSE slope (20kN/m long term strength at 0.6m vertical intervals, varying between approx. 12m-15m long). Gully mullock undercut and replaced with engineered fill.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	[redacted]	1.55	
	Seismic (Stability enhancement)	[redacted]	<1	
K	Long Term Groundwater Conditions	[redacted]	<1	FoS less than the required values for all cases for slip surfaces encroaching within the fill embankment. Stability enhancement measures required. Groundwater at the surface in the gully for all cases.
	Temporary Elevated Groundwater Conditions	[redacted]	<1	
	Seismic	1.0	<1	
	Long Term Groundwater Conditions (Stability enhancement)	[redacted]	1.10	Adequate FoS achieved with inclusion of stability enhancement measures. The minimum FoS for all cases occurs outside the embankment. FoS for slip surfaces encroaching within the fill embankment are above target values, which is acceptable. Proposed Enhancement: Geogrid MSE slope (20kN/m long term strength at 0.6m vertical intervals, varying between approx. 12m long). Gully mullock undercut and replaced with engineered fill.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	1.3	1.10	
	Seismic (Stability enhancement)	1.0	<1	

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
L	Long Term Groundwater Conditions	1.5	1.42	FoS less than the required values for all cases. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	<1	
	Seismic	1.0	1.44	
	Long Term Groundwater Conditions (Stability enhancement)	1.5	1.99	Adequate FoS achieved with inclusion of stability enhancement measures. Proposed Enhancement: Cantilever type retaining/palisade wall (1.5m spacing, 100kN shear capacity per pile, piles approx. 7m long) located at the step between building platforms and on the eastern boundary
	Temporary Elevated Groundwater Conditions (Stability enhancement)	[redacted]	1.36	
	Seismic (Stability enhancement)	[redacted]	>2	
M	Long Term Groundwater Conditions	[redacted]	1.44	FoS less than the required values for all cases for slip surfaces encroaching within the fill embankment. Stability enhancement measures required. Groundwater at the surface in the gully for all cases.
	Temporary Elevated Groundwater Conditions	[redacted]	<1	
	Seismic	1.0	<1	
	Long Term Groundwater Conditions (Stability enhancement)	[redacted]	1.47	Adequate FoS achieved with inclusion of stability enhancement measures for long term and temporary elevated groundwater conditions. Some slip surfaces over the eastern natural slope are below the target values but these don't encroach to be within the lots. FoS less than 1 for seismic conditions but deformations calculated using the Jibson (2007) method are between 0 and 3mm, which is considered to be acceptable. Proposed Enhancement: Geogrid MSE slope (20kN/m long term strength at 0.6m vertical intervals, approx. 5m long).
	Temporary Elevated Groundwater Conditions (Stability enhancement)	1.3	1.35	
	Seismic (Stability enhancement)	1.0	<1	

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
N	Long Term Groundwater Conditions	1.5	1.12	FoS less than the required values for all cases for slip surfaces occurring within the steep batter adjacent to the northern boundary. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	<1	
	Seismic	1.0	<1	
	Long Term Groundwater Conditions (Stability enhancement)	1.5	1.86	Adequate FoS achieved for all scenarios with inclusion of stability enhancement measures Proposed Enhancement: Two 3m high retaining walls (1.5m spacing, 200kN shear capacity per pile, piles approx. 11m long for the upper wall, 9m long for the lower wall) along the northern boundary with an 18 degree slope in between them.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	[redacted]	1.3	
	Seismic (Stability enhancement)	1.0	1.23	
O	Long Term Groundwater Conditions	1.5	1.27	FoS less than the required values for all cases for slip surfaces occurring within the steep batter adjacent to the northern boundary. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	[redacted]	<1	
	Seismic	1.0	<1	
	Long Term Groundwater Conditions (Stability enhancement)	[redacted]	>2	Adequate FoS achieved for all scenarios with inclusion of stability enhancement measures Proposed Enhancement: One 4.3m high retaining wall (1.5m spacing, 280kN shear capacity per pile, piles approx. 9m long) with an 18 degree slope above sloping up to the northern boundary.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	1.3	1.44	
	Seismic (Stability enhancement)	1.0	1.36	

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
P	Long Term Groundwater Conditions	1.5	1.71	FoS less than the required values for temporary elevated groundwater conditions for the batter slope above the WWTP. Stability enhancement measures required. Adequate FoS are available for all scenarios for slope below the effluent disposal trench.
	Temporary Elevated Groundwater Conditions	1.3	1.09	
	Seismic	1.0	1.10	
	Long Term Groundwater Conditions (Stability enhancement)	1.5	>2	Adequate FoS achieved for all scenarios with inclusion of stability enhancement measures Proposed Enhancement: A 2.7m high retaining wall (1.5m spacing, 100kN shear capacity per pile, piles approx. 6m long) with an 18 degree slope above sloping up to the south western boundary.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	[redacted]	1.70	
	Seismic (Stability enhancement)	[redacted]	1.09	

[redacted]

[redacted]

[redacted]

[redacted]

Table 9: Summary of Stability Analysis Results – Post Development (Stage 2)

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
Q	Long Term Groundwater Conditions	1.5	<1	FoS less than the required values for long term and temporary elevated groundwater cases. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	<1	
	Seismic	1.0	<1	
	Long Term Groundwater Conditions (Stability enhancement)	1.5	>2	Adequate FoS achieved for slip surfaces encroaching into the lots (i.e. FoS less than the target values occur outside the lots), with inclusion of stability enhancement measures for long term and temporary elevated groundwater conditions. Proposed Enhancement: Palisade wall (1.5m spacing, 120kN shear capacity per pile, piles approx. 7m long) below the toe of the engineered fill with geogrid MSE slope (20kN/m long-term strength at 0.6m vertical intervals, approx. 3m long) above.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	[redacted]	>2	
	Seismic (Stability enhancement)	1.0	1.4	
R	Long Term Groundwater Conditions	1.5	<1	FoS less than the required values for all cases. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	<1	
	Seismic	[redacted]	<1	
	Long Term Groundwater Conditions (Stability enhancement)	[redacted]	>2	Adequate FoS achieved with inclusion of stability enhancement measures. Proposed Enhancement: Slope above the stormwater pond supported by soil nails (0.8m spacing, 50kN tensile capacity, upper four rows 8m long, lower rows 6m long). Fill slope below the pond supported by a palisade wall (1.5m spacing, 120kN shear capacity per pile, piles approx. 3.5m long) with geogrid MSE slope (20kN/m long-term strength at 0.6m vertical intervals, approx. 5m long) undercut to approximatelly 1.2m depth below the toe of the fill.
	Temporary Elevated Groundwater Conditions	1.3	1.39	
	Seismic (Stability enhancement)	1.0	1.25	

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
S	Long Term Groundwater Conditions	1.5	>2	FoS adequate for all cases. Stability enhancement measures not required.
	Temporary Elevated Groundwater Conditions	1.3	>2	
	Seismic	1.0	1.59	
T	Long Term Groundwater Conditions	1.5	1.41	FoS less than the required values for temporary elevated groundwater conditions. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	<1	
	Seismic	[redacted]	1.01	
	Long Term Groundwater Conditions (Stability enhancement)	[redacted]	1.75	Adequate FoS achieved with inclusion of stability enhancement measures. Proposed Enhancement: Palisade wall (1.5m spacing, 150kN shear capacity per pile, piles approx. 8m long) below the toe of the engineered fill with geogrid MSE slope (20kN/m long term strength at 0.6m vertical intervals, approx. 6m long) above. Counterfort drains also to be installed behind the MSE slope to approx. 4m depth, and 10-15m long, at a 15-20m spacing.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	1.3	1.32	
	Seismic (Stability enhancement)	1.0	1.09	
U	Long Term Groundwater Conditions	[redacted]	1.30	FoS less than the required values for temporary elevated groundwater conditions. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	[redacted]	<1	
	Seismic	1.0	<1	
	Long Term Groundwater Conditions (Stability enhancement)	1.5	1.60	

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
	Temporary Elevated Groundwater Conditions (Stability enhancement)	1.3	1.36	Adequate FoS achieved with inclusion of stability enhancement measures. Proposed Enhancement: 4.4m high retaining wall (1.5m spacing, 120kN shear capacity per pile, piles approx. 9m long) with an 18 degree batter sloping up to the site boundary. Engineered fill batter within the lower portions of the slope to comprise a MSE slope (20kN/m long term strength at 0.6m vertical intervals, approx. 8m long with an undercut of approx. 2.5m).
	Seismic (Stability enhancement)	1.0	1.13	
V	Long Term Groundwater Conditions	1.5	<1	FoS less than the required values for all ground water conditions. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	[redacted]	<1	
	Seismic	[redacted]	<1	
	Long Term Groundwater Conditions (Stability enhancement)	1.5 [redacted]	1.54	Adequate FoS achieved with inclusion of stability enhancement measures. Some slip surfaces over the southern natural slope are below the target values but these don't encroach to be within the lots. Proposed Enhancement: Adjacent to the south facing slope install a palisade wall (1.5m spacing, 200kN shear capacity per pile, piles approx. 13.5m long). For the northern fill slope an MSE slope is required (40kN/m long term strength at 0.6m vertical intervals, the upper 6 rows of geogrid to be approx. 20m long with the balance being approx. 16m long. A palisade wall (1.5m spacing, 300kN shear capacity per pile, piles approx. 8m long) is required at the toe of the fill batter. Counterfort drains also to be installed below the MSE slope to approx. 5m depth and extending 15 to 20m upslope of the MSE slope, at a 10-15m spacing.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	[redacted]	1.44	
	Seismic (Stability enhancement)	[redacted] 1.0 [redacted]	1.06	

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
W	Long Term Groundwater Conditions	1.5	<1	FoS less than the required values for all ground water conditions. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	<1	
	Seismic	1.0	<1	
	Long Term Groundwater Conditions (Stability enhancement)	1.5	1.89	Adequate FoS achieved with inclusion of stability enhancement measures. Proposed Enhancement: Palisade wall (1.5m spacing, 300kN shear capacity per pile, piles approx. 5-7m long) installed at the base of each fill batter with and MSE slope above (20kN/m long term strength at 0.6m vertical intervals, approx. 5m long. The upper slope excavated down to the platform level to facilitate MSE construction. Lower MSE slope with an undercut of approx. 2m).
	Temporary Elevated Groundwater Conditions (Stability enhancement)	[redacted]	1.33	
	Seismic (Stability enhancement)	[redacted]	1.24	
X	Long Term Groundwater Conditions	[redacted]	1.20	FoS less than the required values for all groundwater conditions. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	[redacted]	<1	
	Seismic	1.0	<1	
	Long Term Groundwater Conditions (Stability enhancement)	[redacted]	1.61	Adequate FoS achieved with inclusion of stability enhancement measures. Proposed Enhancement: Palisade wall (1.5m spacing, 300kN shear capacity per pile, piles approx. 8m long) with geogrid MSE slope (20kN/m long term strength at 0.6m vertical intervals, approx. 10m long). Counterfort drains also to be installed below the MSE slope to approx. 5m depth, daylighting below the MSE slope and extending 15m to 20m upslope of the MSE slope, at a 10m-15m spacing.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	1.3	1.38	
	Seismic (Stability enhancement)	1.0	1.07	

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
Y	Long Term Groundwater Conditions	1.5	<1	FoS less than the required values for all cases. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	<1	
	Seismic	1.0	<1	
	Long Term Groundwater Conditions (Stability enhancement)	1.5	>2	Adequate FoS achieved with inclusion of stability enhancement measures for all cases. FoS less than 1 for seismic conditions but deformations calculated using the Jibson (2007) method are less than 2mm, which is considered to be acceptable. Proposed Enhancement: All fill batters to include geogrid MSE slope (20kN/m long term strength at 0.6m vertical intervals, approx. 3m-5m long). Palisade wall to be located at the base of the near vertical fill slopes (1.5m spacing, 100kN shear capacity per pile, approx. 5m-6.5m long). All near vertical cuts to be supported by cantilever type retaining walls (1.5m spacing, 100kN shear capacity per pile, approx. 8m long)
	Temporary Elevated Groundwater Conditions (Stability enhancement)	[redacted]	1.51	
	Seismic (Stability enhancement)	[redacted] 1.0 [redacted]	<1	
Z	Long Term Groundwater Conditions	[redacted]	<1	FoS less than the required values for all cases for slip surfaces encroaching on the platform. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	<1	
	Seismic	[redacted]	<1	
	Long Term Groundwater Conditions (Stability enhancement)	[redacted]	>2	Adequate FoS achieved for slip surfaces encroaching within the platform area with inclusion of stability enhancement measures. Proposed Enhancement: Proposed Enhancement: Palisade wall on the downslope edge of the platform (1.5m spacing, 200kN shear capacity per pile, piles approx. 7m long).
	Temporary Elevated Groundwater Conditions (Stability enhancement)	1.3	1.40	

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
	Seismic (Stability enhancement)	1.0	1.18	
AA	Long Term Groundwater Conditions	1.5	<1	FoS less than the required values for all cases. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	<1	
	Seismic	1.0	<1	
	Long Term Groundwater Conditions (Stability enhancement)	[redacted]	1.57	Adequate FoS achieved with inclusion of stability enhancement measures. Proposed Enhancement: Geogrid MSE slope (40kN/m long-term strength at 0.6m vertical intervals, varying between varying from approx. 8m-14m long). Approx 6m deep, 15m wide shear key required below the MSE slope. All steep cut batters to be supported by cantilever retaining walls (1.5m spacing, 100kN shear capacity per pile, piles approx. 5m long).
	Temporary Elevated Groundwater Conditions (Stability enhancement)	[redacted]	1.44	
	Seismic (Stability enhancement)	[redacted]	1.12	
AB	Long Term Groundwater Conditions	[redacted]	1.22	FoS less than the required values for all cases. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	<1	
	Seismic	[redacted]	<1	
	Long Term Groundwater Conditions (Stability enhancement)	[redacted]	1.50	Adequate FoS achieved with inclusion of stability enhancement measures. Proposed Enhancement: Geogrid MSE slope (20kN/m long-term strength at 0.6m vertical intervals, approx. 10m long). Approx 5m deep, 10m wide geogrid reinforced shear key required below the MSE slope.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	1.3	1.43	

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
	Seismic (Stability enhancement)	1.0	1.06	All steep cut batters to be supported by cantilever retaining walls (1.5m spacing, 100kN shear capacity per pile, piles approx. 5m long). Counterfort drains also to be installed behind the MSE slope to approx. 5m depth, extending 15 to 20m upslope of the MSE slope, at a 10m-15m spacing.
AC	Long Term Groundwater Conditions	1.5	1.19	FoS less than the required values for all cases for slip surfaces. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	<1	
	Seismic	1.0	<1	
	Long Term Groundwater Conditions (Stability enhancement)	[redacted]	1.51	Adequate FoS achieved with inclusion of stability enhancement measures. Proposed Enhancement: Soil nails (1.2m spacing, 50kN tensile capacity, 5m long) over the upper soil portion of the cut batter adjacent to the western boundary.
	Temporary Elevated Groundwater Conditions (Stability enhancement)	[redacted]	1.41	
	Seismic (Stability enhancement)	[redacted]	1.04	
AD	Long Term Groundwater Conditions	1.5	1.08	FoS less than the required values for all cases. Stability enhancement measures required.
	Temporary Elevated Groundwater Conditions	1.3	<1	
	Seismic	[redacted]	<1	
	Long Term Groundwater Conditions (Stability enhancement)	1.5	1.89	

Cross-Section	Scenario	Target FoS	Minimum Achieved FoS	Comments
	Temporary Elevated Groundwater Conditions (Stability enhancement)	1.3	1.39	Adequate FoS achieved for all scenarios with inclusion of stability enhancement measures. Proposed Enhancement: Geogrid MSE slope (20kN/m long term strength at 0.6m vertical intervals, varying approx. 6m long). Palisade wall below the MSE slope (1.5m spacing, 280kN shear capacity per pile, piles approx. 10m long). Counterfort drains also to be installed from approximately 10m below the toe of the MSE slope to approx. 5m depth, extending 15m to 20m upslope of the MSE slope, at a 10m-15m spacing.
	Seismic (Stability enhancement)	1.0	1.19	

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

5.1.5 Discussion on Slope Stability

The slope stability analyses results show an unacceptable FoS against instability is available across significant portions of the site, typically as a result of the combination of saturated groundwater conditions and steep proposed slopes. The analysis results indicate that the existing instability features are likely caused by saturated groundwater conditions. Accordingly, the control of both surface water and groundwater is important.

As mentioned in earlier sections, a several existing instability features have been identified that will need to be remediated. The measures considered to be required to ensure that adequate FoS against instability are outlined in Tables 8 and 9 above and discussed below.

5.1.6 Stability Enhancement Measures

In order to provide an adequate FoS against slope instability for the building lots, the associated accessways and batters, we conside[redacted]ing concept stability enhancement measures need to be incorporated into the subdivision design:

- **Subsoil drainage** beneath all [redacted]configuration including cross bench, main lines and herringbone laterals) to minimise the risk of soil profile saturation and provide a degree of control over groundw[redacted]
- All fill slopes steeper than 1v in [redacted] **geogrid reinforcement** (e.g. mechanically stabilised earthfill, MSE) of varying lengths included. Most fill batters that are on the edges of the gullies are approximately 45 degrees. At some locations an inground **palisade wall** is required to be installed bel[redacted]of the MSE slope.
- For perimeter cut batters i[redacted] eastern and southern parts of the site, steeper than 1v in 3h **soil nails** are [redacted] be a suitable measure, subject to there being sufficient space to construct the **soil nails**. As an alternative, a cantilever type **retaining wall** could be constructed to a height sufficient to reduce the batter gradient to 1v in 3h. Depending on the height of the retaining wall, tiebacks may be required.
- For the areas in the vicinity of cross-sections A, I, and M within the central portion of Stage 1 a suitable option is to excavate the unstable ground and substantially rework the slope to form a **shear key** with **subsoil drainage. Geogrid** reinforcement will also need to be incorporated in the fill batters.
- Specifically for cross-section A, **palisade retaining walls** are required to support the mid and upper fill platforms.
- Two tiered **retaining walls** are required to support the northern boundary cut.
- Internal batters between lot platforms are to be supported by **cantilever or gravity type retaining walls**.
- For retaining/palisade locations where the required unfactored shear capacity (see Table 8 above) is less than 150kN, 400mm-450mm timber SED piles should be satisfactory. For **retaining/palisade walls** that the stability analysis indicates require an unfactored shear capacity of 200kN, 280kN or 300kN, steel piles similar to 310UC97's or steel reinforced concrete piles likely the range of 600mm to 750mm-diameter will likely be required. These indicative pile sizes will need to be reviewed and refined during detailed design.

- At some locations **counterfort** (e.g. trench type) drains are required to either below and/or above MSE slopes (this is predominantly within Stage 2) to provide control over ground saturation. These drains are typically 5m deep and at 10m-15m spacing and typically in the order of 20m long.
- **Shear keys** (with and without geogrid reinforcement) are also proposed below the MSE slopes at several locations, predominantly within Stage 2. The shear keys are indicated to be 5m-6m deep and 10m to 15m wide and extend into the underlying rock mass.

The preliminary proposed locations and extents of these measures are indicated on the appended Sketches SK140 to SK144. The general conceptual configuration of these measures is also indicated on the appended sketches. The in-ground walls, soil nails, MSE slopes, retaining walls and earthworks measures (e[redacted]keys and subsoil drainage) will require specific geotechnical input to the detailed design. In addition, all other batters steeper than 1v in 4h will require further geotechnical input d[redacted]design. The final extent and locations of the proposed remedial stability meas[redacted] to be confirmed on-site by Riley during construction.

In addition to these measures, it is al[redacted] surface water is controlled to ensure that all runoff from impervious areas is collected and piped to suitable locations remote from the building platforms and accessways. [redacted]ischarges of stormwater onto steep slopes should be avoided. Recommended [redacted]charge stormwater are discussed further in Section 5.9.

Furthermore, we recommend tha[redacted] possible, the existing vegetation should be supplemented with new plantings and the steep proposed slopes should also be vegetated. The contribution of vegetation to overall ground stability should not be underestimated. From review of the Greenwood Associates pr[redacted]getation Plan we understand that substantial re-vegetation of the steeper slopes adjacent to the streams is proposed. This is consistent with our recommendation above.

5.2 Groundwater Impact Assessment

An assessment has been made of the impact of the proposed development on groundwater conditions in accordance with the requirements of Section E7 of the AUP. The assessment considers the impact of the development proposal for groundwater diversion activities. The results are contained within Table 10 below. The results are also discussed below.

As outlined in Section 2.5, the site contains a series of ridges and gullies. We consider that this topography has led to the formation of localised groundwater regimes between the gullies. Further, groundwater level monitoring to date indicates that groundwater is likely perched on top of the shallow rock that is present throughout the site. The proposed earthworks generally involve cutting from the elevated ridge lines and filling on their side slopes as well as across the gullies to construct accessways. Subsoil (underfill) drains will be installed as part of the earthworks.

Any groundwater intercepted by these will be returned to the streams/wetlands in the gullies and will not be diverted to other catchments. As such, we consider that the proposed excavations should not alter the receiving flows for the downstream catchments. Accordingly, for the bulk of the development there should be no groundwater drawdown effects that extend beyond the site boundaries with respect to the downstream receiving environments.

There are some areas where it is proposed to form cut batters adjacent to the external boundary of the development and where the groundwater table is likely to be intercepted. These are primarily proposed along the northern and eastern boundaries of Stage 1 where cuts of up to approximately 9m and 8m respectively are proposed. There are also some smaller cuts of approximately 4-5m along the southern boundary of Stage 1. Within Stage 2 there are cuts of up to 14m mid-way along the western boundary and 12m in the northern end of the eastern boundary. Elsewhere, where there are boundary cuts on the eastern and southern boundaries, they are in the range of 8m-9m depth.

Based on review of the groundwater level monitoring data collected to date, we consider that the boundary cuts will exceed the permitted activity rules in relation to E7 of the AUP. See Table 10.

Table 10: Auckland Unitary Plan Chapter E – Auckland Wide, Part 7 Taking, using, Damming and Diversion of Water and Drilling [redacted]

Rule		Activity	Applicability to this Site
E7.6.1.6	Dewatering or groundwater level control associated with a groundwater diversion permitted under Standard E7.6.1.10, all of the followin[redacted] must be met – Permitted Activities	[redacted]ater take must not be geothermal water; • The [redacted] take must not [redacted] period of more than ten days where it [redacted] in peat soils, or 30 days in other types of soil or rock; and [redacted] The water take must only occur during construction.	Does not comply. Diversion is permanent.
E7.6.1.10.(1)	Diversion of groundw[redacted] caused by any excav[redacted] (including trench) or tunnel - Permitted Activities	[redacted]e following activities [redacted]mpt from the Standards E7.6.1.10(2) – (6): • Pipes cables or tunnels including associated structures, which are drilled or thrust and are less than 1.2m in external diameter; • Pipes including associated structures up to 1.5m in external diameter where a closed faced or earth pressure balanced machine is used; • Piles up to 1.5m in external diameter are exempt from these standards; • Diversions for no longer than ten days; or • Diversions for network utilities and road network linear trenching activities	Does not comply: Permanent diversion due to groundwater drawdown for proposed site earthworks

Rule		Activity	Applicability to this Site
		that are progressively opened, closed and stabilised where the part of the trench that is open at any given time is no longer than ten days.	
E7.6.1.10.(2)		Any excavation that extends below natural groundwater level, must not exceed: • 1ha in total area; and [redacted]m depth below the [redacted]atural ground level [redacted]	**Does not comply:** The total cut area is greater than 1ha. The maximum depth of excavation is locally between 4m and 14m below the natural ground level where cuts are proposed adjacent to the property boundary.
E7.6.1.10.(3)		[redacted] groundwater level must not be reduced by more [redacted] the boundary of the adjoining site [redacted] [redacted]	Does not comply: Our assessment indicates a for Stage 1 a worst-case total drawdown of the groundwater table of 6m at cuts along the northern boundary, 5m in cuts in the northern portion of the eastern boundary and 2m-3m in the cuts along the southern portion of the eastern boundary and southern boundary. For Stage 2 the worst case groundwater drawdown of 6m occurs along the southern boundary. Elsewhere, due to the proximity of rock to the ground surface the groundwater drawdown is limited to the top of the rock. The deepest being 5.2m (for approximately 3.2m of groundwater drawdown) along the southern part of the eastern boundary adjacent to Ara Hills.

Rule		Activity	Applicability to this Site
E7.6.1.10.(4)		Any structure, excluding sheet piling that remains in place for no more than 30 days, that physically impedes the flow of groundwater through the site must not: • Impede the flow of groundwater over a length of more than 20m; and • extend more than 2m below the natural [redacted]roundwater level.	Complies: The required stability enhancement measures should not impede groundwater flows.
E7.6.1.10.(5)		The distance to any existing [redacted] or structure (excluding timber fences and small structures on the [redacted] an adjoining site from the edge of any: • [redacted] or open excavation that extends below natural groundwater level must be at least equal to the [redacted]epth of the excavation; • tunnel or pipe with an [redacted]ernal diameter of 0.2 – 1.5m that extends below natural groundwater level must be 2m or greater; or • a tunnel or pipe with an external diameter of up to 0.2m that extends below natural groundwater level has no separation requirement.	Partly Complies: The minimum distance from the Stage 1 bulk excavation to the nearest structure is >20m (greater than 9m maximum depth of excavation adjacent to the site boundary). The nearest structure considered is an adjacent consented road for Stages 6/7 of the AV Jennings Ltd Ara Hills Development. All other existing dwellings are located at least 40m away from the boundary cuts. For Stage 2 a farm shed is adjacent to the southern boundary. This shed is approximately 6m from the closest excavation and is in proximity to the maximum excavation along the southern boundary. Elsewhere around the perimeter of Stage 2 the nearest structure considered is an adjacent consented road for Stages 6/7 of the AV Jennings Ltd Ara Hills Development which is approximately 20m form the edge of the cuts on the eastern boundary. There are no

Rule		Activity	Applicability to this Site
			other existing structures located within 40m of the boundary cuts.
E7.6.1.10.(6)		The distance from the edge of any excavation that extends below natural groundwater level, must not be less than: • 50m from the Wetland Management Areas Overlay; [redacted]0m from a scheduled [redacted]istoric Heritage Overlay; or [redacted]rom a lawful groundwater take.	Complies: The excavation edge is in excess of 50m from any wetland management overlays, greater than 10m from scheduled historic heritage overlays and lawful groundwater takes.

The ground conditions comprise a com[redacted]ed fill, NA and ECBF rock. Based on experience and published literature, a permeability [redacted]/s is considered to be representative of the soils present in Stage 1. For Stage 2 a perm[redacted]f 1 x 10^{-6} m/s was adopted to account for the presence of materials with increased silt/sand contents and lower plasticity.

Groundwater measurements indicat[redacted]proposed excavations could induce groundwater drawdown of up to 6m in the vicinity of the northern boundary of Stage 1 and southern boundary of Stage 2. We have assessed th[redacted]of the associated settlement at the location of maximum groundwater drawdow[redacted]he base of the deepest excavations adjacent to the site boundary). The extent of groundwater drawdown has been assessed in accordance with the CIRIA C515 (Groundwater Control – Design and Practice) document. The calculated settlements for in the vicinity of the northern boundary of Stage 1 are up to 14mm and the calculated maximum extent of the influence is up to approximately 5.4m. However, because of the slope of the cut batters, the location of maximum drawdown is more than 5.4m inside the site boundary. Similarly for Stage 2 the maximum calculated settlement is up to 14mm at the base of the batter along the southern boundary. However, as for Stage 1 above, because of the batter slope, the extent of groundwater drawdown is calculated to be within the site (e.g. 18m from the toe of the batter which is 25m from the boundary).

In any event the farm shed that is present near the southern boundary is beyond the extent of groundwater drawdown. Elsewhere around the western and northern and eastern boundaries of Stage 2, at locations where excavations extend up to the boundary, we anticipate that there will be some localised groundwater drawdown that could extend a short distance beyond the site boundary but there are no existing structures within the zone of influence that are likely to be affected.

Accordingly, we consider that there should be no influence on the groundwater table extending beyond the site boundary except as outlined above. There are no existing structures within the zone of influence and therefore the drawdown effect on neighbouring sites is expected to be negligible. Calculations are appended in Appendix F. We also consider that due to the limited extent of the groundwater drawdown, there should be no adverse effect on the groundwater source.

5.3 Liquefaction and Lateral Spreading

The age, consistency, composition, and stress history of the soil and groundwater conditions (discussed above) are not consistent with soils that are prone to liquefaction and lateral spreading. Therefore, liquefaction and lateral spreading is considered to be unlikely to occur here during a ULS seismic event. [redacted]

5.4 Building Foundations [redacted]

Away from the gully inverts, the relatively stiff natural soils have been encountered. We consider that these stiff soils and the enginee[redacted]ced as part of subdivisional development works should be suitable to provide a geotechnical ultimate bearing capacity of 300kPa for the design of conventional shallow-type [redacted] foundations for NZS 3604 type residential structures up to three levels high [redacted]ore than 5m from slopes with gradients exceeding 1v in 4h, except where dwellings are located above a MSE slope in which case a setback of 3m is likely to be required. This will need to be confirmed during detailed design and subsequent geotechnical completio[redacted]g. The stability analyses indicate that following construction of the stability enhancement measures no specific Building Restriction Line is required. This will need to be reviev[redacted]paration of the Geotechnical Completion Report following completion of the site de[redacted]rthworks.

Specific site investigation and foundation design will be required for all structures that extend downslope of the BRL or are within 5m of land with gradients exceeding 1v in 4h. This will be confirmed in a geotechnical completion report to be prepared following completion of site earthworks.

From review of the laboratory test results, our preliminary assessment of the expansive soil class for lots underlain by Northland Allochthon soils (either natural or fill) is High to Extreme in terms of AS 2870. Areas underlain by soils of the ECBF deposits are likely to be Moderately to Highly expansive. Accordingly, building foundations should be designed in accordance with B1/AS1 or AS 2870 provided that in the former case the foundation depths are consistent with those recommended in these standards. We consider that required foundation depths are likely to be in the range of 600mm to 900mm, for foundations designed in accordance with B1/AS1 dependent on the final assessed expansive site classifications across the development. Alternatively, a specifically designed raft-type foundation system could be utilised.

Further testing is recommended during preparation of the GCR following site earthworks to delineate areas of high expansivity.

5.5 Retaining Walls

In addition to the measures required for stability enhancement as discussed in Section 5.1.6, wherever near vertical batters greater than 0.6m in height are proposed between adjacent building platforms, they should be supported by specifically design retaining walls.

For specific design of the walls, we envisage that the following parameters may be used:

- Ka (active) earth pressure modified for ground slope behind for free standing walls.
- K_0 (at rest) earth pressure coefficients should be modified for slopes and surcharges for all walls in close proximity to future structures, driveways, or near proposed lot boundaries.
- Retained Soils: Refer Table 7 i[redacted]1.3 for soil effective stress parameters.
- Embedment: Refer Table 7 in [redacted] soil effective stress parameters. or Su= 50kPa for Brom's solution.
- Allowance for building and boundary surcharge loading as applicable.
- The retaining walls should be c[redacted]propriate toe drainage and backfilled to their full height with lightly tamped, granular material that complies with the TNZ F/2 specification. Toe drainage should be connect[redacted]oved stormwater disposal system.

These values should be appropriate for the soils encountered. However, if significant zones of soft strata are exposed during the excav[redacted] designs should be revised, and Riley should be contacted for further advice.

Groundwater is likely to be encou[redacted]construction, accordingly, allowance should be made for the use of sumps and pumps.

5.6 Earthworks

The drawings provided to us indicate that earthworks comprising cuts and fills up to 16m and 15m respectively are proposed. These are primarily associated with easing of site gradients and the formation of associated roads. Where design batter gradients are steeper than 1v in 4h further geotechnical input will be required during detailed design.

An earthworks specification should be prepared to assist the earthworks contractor. The key elements are broadly outlined below.

5.6.1 Stripping and Site Preparation

All topsoil, organic soils and non-engineered fill should be stripped from the proposed earthworks areas, and either stockpiled (well clear of the earthworks) for re-spreading (if suitable) at the completion of earthworks or removed from site. Where topsoil is re-spread its depth should be less than 300mm. All debris from demolition of the existing structures should be removed from site and the subgrade inspected by a geotechnical professional familiar with the contents of this report to assess if any undercutting is required.

Our investigations to date indicate that soft and/or organic soils are present to depths typically in the range of 1.5m (HA107, eastern part of Stage 1) to 4m (HA135, north-western corner of Stage 1) in the low-lying parts of the site, specifically in the base of gullies. These will need to be undercut at locations where culverts and/or fills are proposed. Prior to placement of any fill, a geotechnical professional familiar with the contents of this report, should inspect the formed subgrade to observe the exposed soils and assess if further undercutting is required. The excavated material should be replaced with earth fill, placed and compacted in accordance with Section 5.6.5.

5.6.2 Cut and Fill Materials

The site cuts will predominantly comprise Northland Allochthon and Waitemata Group soils that are present across the site. These should be suitable, with conditioning for use as engineered fill. Due to the variable soil plasticity and [redacted]ent of the soils, some of the siltier soils are likely to be difficult to earthwork. It is also likely that weathered sandstones and siltstones will make up a portion of the site fills. These mate[redacted]cted to break down into predominantly low plasticity soils under earthworks co[redacted]rdingly, it will be important that soils of a high plasticity are blended with the more silty/sandy soils to assist with placement and compaction of earthworks fill consistent with the req[redacted]431. In this regard it will be important that the earthworks contractor's methodology gives consideration to the distribution of high and low plasticity soils across the developme[redacted]

Although not encountered during our investigations, any pumiceous soils encountered should be excluded from the earthworks. The suitability of any existing fill for inclusion in the site earthworks should be determined on-site by a [redacted]eotechnical Engineer/Geologist familiar with the contents of this report.

Based on the groundwater leve[redacted]o date, we expect that groundwater will be encountered during site earthworks cuts. We envisage the Contractor should be able to suitably manage groundwater inflows through the construction of subsoil drainage, in combination with the use of temporary sumps and pumps. In any event, the contractor will need to ensure that their earthworks methodology allows for the interception of the groundwater table. We would expect this to be included in the Contractors Construction Management Plan.

As outlined above, moisture content conditioning will likely be required to dry back the fill materials to enable compaction at the OMC. Care will need to be taken to not over-dry the fill material. The earthworks contractor should be aware that the laboratory test results indicate the OMC for NA fill is approximately 5-15% higher than the OMC for the Waitemata Group soils. Accordingly, the NA fill materials should not be conditioned to the same moisture content as the WG materials. If over-drying occurs, then the materials should be wet back up to optimum moisture content. From the laboratory test results, we anticipate that conditioning may require alteration of the moisture content by up to approximately 20%. While we anticipate that conditioning will primarily be achieved through discing and mixing, lime/cement stabilisation is also expected to be suitable enable the soils to be placed to the required compaction criteria.

5.6.3 Rock Excavation

The cuts beyond 2.0m depth in the Northland Allochthon zone and 2.8m depth in the Waitemata Group zone could encounter weak weathered rock comprising weathered sandstones and siltstones. The approximate extent of rock that is likely to be exposed at finished level is shown on the appended Sketch SK130 in Appendix H.

Test pits were undertaken with a 12 to 13-tonne excavator with a 900mm wide general-purpose rock bucket which refused within the bedrock in the less weathered sandstone and siltstone bedrock of the Waitemata Group and Northland Allochthon. The proposed earthworks plan involves the excavations well below the test pit depths. The results of the machine boreholes give an indication of the character of the [redacted] materials below the test pit depths.

Considering the investigation data, [redacted]ent is that the materials should be readily excavated using conventional eart[redacted] (e.g. excavators with rock buckets and/or bulldozers with rippers). However, the Contractor should make their own assessment based on their envisaged earthworks methodo[redacted]ry they have available.

Following excavation, these materials [redacted]reakdown under compaction and should be suitable for inclusion within the engine[redacted]ed it is appropriately conditioned and mixed with materials of sufficient plasticity.

5.6.4 Cut Subgrade Protection

Where Northland Allochthon rock [redacted] the surface or immediately beneath topsoil, the rock may be subject to rapid we[redacted]egradation, due to dilation of defects resulting from stress relief. The exposure of [redacted]also result in increasing surface water infiltration, due to the high permeability of the rock mass relative to the residual soils. This can result in a reduction of the available FoS against instability in the vicinity.

To mitigate this, wherever Northland Allochthon rock is present within 0.6m to 1m depth below cut earthworks levels, it should be undercut and replaced with a clean compacted clay fill cap. The undercuts should be of sufficient depth to ensure that the clay cap has a minimum thickness of 0.6m to 1m. The final thickness will need to be subject to onsite assessment by the geotechnical engineer during earthworks and give consideration to the characteristics of the exposed rock. In areas where ECBF rock is exposed at the finished surface, consideration should be given to over excavating and replacement with compacted clay fill to more readily facilitate installation of private service reticulation (e.g. power, telephone, gas, stormwater, wastewater) and to ensure that where possible dwelling foundations are not directly underlain by rock and soils (e.g. to mitigate potential differential settlements) for the lots. Further geotechnical input will be required as construction works progress.

5.6.5 Compaction

Earthworks fill compaction testing should be undertaken at or in excess of the frequency recommended in NZS 4431. We envisage that earthworks control will be undertaken principally using allowable air voids and shear strength criteria, although maximum dry density and optimum moisture content may also be used.

Compaction of the filling should be carried out to certifiable standards (NZS 4431) with conventional plant and should be under engineering control. The standard compaction test results indicate that the soils should be suitable to be compacted to comply with the compaction criteria of NZS4431. The preliminary fill compaction criteria are shown in Table 11.

Table 11: Fill Compaction Control Crit[redacted]

Material	Test	Average Value Not Less Than	Minimum Single Value Not Less Than
Cohesive Fill	Shear Vane – Undrained Shear Strength	150kPa	120kPa
	Nuclear Densometer – Air [redacted]	8%	10%

Note: Average to be determined as a rolling mean value over 5 consecutive tests

A specific methodology for place[redacted] testing may be required for inclusion of Northland Allochthon/Waitemata Group rock within the engineered fill if the earthworks contractor has difficulty achieving the above co[redacted] criteria. Further laboratory standard compaction testing will be required for fills comprising a mix of soil and rock materials. However, in general, we consider that a similar specification[redacted]esive fill is expected to be suitable.

5.6.6 Benching of Slopes

Benching of slopes should be undertaken in accordance with NZS 4431 prior to commencement of filling. This is particularly important for any earthworks within the areas adjacent to or within the gullies, steep slopes and existing instability features requiring enhancement. We consider that underfill/subsoil drainage will likely be required in these areas. The extent of underfill/subsoil drainage will need to be confirmed onsite during earthworks.

5.6.7 Underfill, Subsoil and Counterfort Drainage

Underfill and subsoil drainage should consist of 160mm-diameter highway grade drain coils with filter sock, encapsulated within an all passing drainage aggregate to the TNZ F/2 specification. The extent of the underfill and subsoil drains will need to be confirmed during detailed design and on-site during subdivisional works. For a preliminary estimate of total subsoil/underfill drain lengths, it can be assumed that a subsoil drain is required along the base of each cut bench for its full length together with associated cross bench, main line, herringbone lateral drains. These should discharge via regularly spaced lateral drains to collector manholes or suitable outfall locations with appropriately designed outfall structures.

For the deeper fills subsoil drains may need to be installed within the fill to manage internal pore water pressures with respect to a potential adverse effect on slope stability and fill induced settlements. This will be considered further during detailed design. See Section 5.7 below for further comment.

They will need to be surveyed included in as-built drawings as part of completion documentation. Typical underfill and counterfort drain details (Sketches SK202 and SK203) are appended in Appendix B.

5.7 Fill Induced Settlements

Deep fills of the nature proposed on this site may be subject to ongoing settlement for a period following completion of the works. D[redacted]omposition of the underlying soils being very stiff to hard and the relatively shallow overburden profile, settlements due to fills are anticipated to be minor in magnitude. Our experie[redacted] that with appropriate gully preparation (e.g. undercutting of soft/organic soils) and subsoil drainage, settlements typically attenuate soon after earthworks are finished.

[redacted]

If there are any locations where fills are proposed over suspected compressible materials and where fill depths exceed 5m, settlem[redacted] points should be installed at the finished surface following the completion of f[redacted]ubject to survey monitoring to confirm that settlement rates have sufficiently attenuated for the proposed development. The inclusion of subsoil drains at 3m-5m vertical intervals within such fills will shorten the drainage path length and thereby reduce the time for fill i[redacted]lement attenuation. For the proposed fill depths, we anticipate that fill induced settlements are likely to be in the order of 25mm-50mm.

The number, positioning and freq[redacted]construction monitoring is to be confirmed with the Geotechnical Engineer during construction.

5.8 Rubbish Pits

The presence of rubbish pits is not uncommon within a farm setting and there may be locations within the development that contain areas of buried rubbish. Where encountered within the development area during site works, they should be excavated and backfilled with clean clay fill, placed and compacted in accordance with NZS 4431. All material excavated from rubbish pits within the development area should be removed from site.

5.9 Stormwater Control

It is important that due care is paid to the design and construction of appropriate stormwater disposal systems. These systems should serve to collect all stormwater runoff from roofs, water tank overflows, decks, driveways and other paved areas, together with discharges from and other subsoil drains. All stormwater discharges should be piped to suitable outfall locations, such as gully bases, ponds and creeks etc.

Stormwater dispersal/soakage (e.g. raingardens, swales, soak pits) and outfall structures should be designed by a chartered progressional engineer experienced in stormwater design and familiar with the contents of this report. They would typically be lined. This is beneficial from a geotechnical standpoint with respect to slope stability. Stormwater soakage into Northland Allochthon soils is not recommended due to potential effects on the underlying rock mass and local stability. Where stormwater devices are excavated into Northland Allochthon materials, care will need to be taken to ensure that there is 0.6m to 1m thick clay cap over the underlying rock mass. In any event further geotechnical input will be required during detailed design of such devices to ensure that adequate FoS against instability are maintained.

For lots that are adjacent to the local gullies, discharge of stormwater over the engineered fill batters should be suitable, provided that the discharge from individual lots is via an approved energy dissipation device and flow ra[redacted]ficiently low to prevent scour of the batter surface. To this end we recommend that erosion protection is installed with geotechnical input. This will be reviewed during detailed design. [redacted]

We have reviewed the available McKenzie and Co Ltd drawings and consider that they are generally in alignment with our recon[redacted]ve with respect to stormwater control.

5.10 Effluent Disposal [redacted]

The field investigation findings and laboratory PSD test results indicate that the natural soils present at the site are generally consistent with soil category 5 as described in the Auckland Council guideline document for design of o[redacted]water management - GD06. This indicates an application rate of 8mm-12mm/day for trench type application and 2mm-3mm/day for pressure compensating dripper lines, for efflu[redacted]een subject to secondary treatment.

The available investigation data indicates the groundwater table at the time of drilling was beyond 3m depth (i.e. at least 1m below the base of the discharge trenches). Underfill drainage upslope of the discharge trenches should also assist with controlling the groundwater level.

The on-site effluent discharge system is being designed by others. However, with respect to maintaining adequate FoS against ground instability we consider that the trench locations proposed by McKenzie & Co Ltd within Stage 1 immediately to the east of the Treatment Plant should be suitable. Dripper irrigation within the bush area in the north-eastern part of Stage 1 should also be suitable with respect to stability. This will be reviewed during detailed design.

5.11 Services

If soils of low plasticity are encountered, they are considered to be susceptible to collapse, erosion and internal piping in trenches excavated below the water table. For services constructed in such soils and where pipe gradients are steep, we recommend that seepage collars should be constructed at regular intervals (i.e. upstream of each manhole) along the service line trenches to prevent the migration of the fine soil fraction along the trenches and associated erosion and subsidence.

The contractor should also expect to encounter Northland Allochthon and or Waitemata Group rock for portions of service pipeline alignment. The contractor should ensure that their construction methodology is suitable for formation of pipe trenches in such ground conditions.

It is recommended that installation of stormwater lines be undertaken utilizing trench shields and/or battering provided the shoring methodology complies with the relevant NZ standards and legislation. The use of sumps and pumps will also likely be required to control groundwater inflows during service line installation.

Where proposed stormwater lines are oriented parallel to contour, it may also be necessary to install a 110mm-diameter perforated drain coil (with sock) in the base of the trenches to ensure that water within the trenches does not adversely affect slope stability. This is important for services in close proximity to the [redacted]es present in parts of the site. In any event geotechnical input will be required during detailed design for this. For much of the service lines, it will also be necessary to ensure th[redacted]ackfill is compacted to normal engineering standards (e.g. NZS 4431). This is particularly important for those service lines that are located in areas where materials of the Northland Allochthon are present. [redacted]

5.12 Roads

We recommend that a programme of [redacted]rtaken during site earthworks to confirm the available CBR at the road subgrade level. It will also be important that the suitable drainage measures are installed to help protect the subgrade. Based on our investigations to date we anticipate that a subgrade CBR [redacted] 4% should be available within the natural Waitemata Group and Northland Allochthon soils exposed at subgrade level, while for engineered clay fill, we would expect a CBR of [redacted] be available. A CBR in excess of 7% should be expected for areas where rock is e[redacted]grade level.

5.13 Inspections

The opinions, recommendations and comments given in this report result from the application of normal methods of site investigation. As factual evidence has been obtained solely from test pits and boreholes, which by their nature only provide information about a relatively small volume of subsoils, there may be special conditions pertaining to this site which have not been disclosed by the investigation, and which have not been taken into account in the report. Considering this and the nature of the ground conditions, It is important that we are given the opportunity of inspecting the site clearing operations, earthworks operations and site drainage works to ensure that the ground conditions encountered are as anticipated from the findings of this report.

If they are not, we would be on hand to recommend the most appropriate design and/or construction modifications.

We would appreciate at least 24 hours' notice prior to site inspections.

Upon satisfactory completion of these aspects of the works, we would then be in a position to issue an appropriate geotechnical completion report.

5.14 Proposed Geotechnical Consent Conditions

The proposed geotechnical consent conditions are appended in Appendix G. These address the geotechnical monitoring and reporting requirements primarily in relation to earthworks and groundwater drawdown associated with the site development works.

6.0 Further Input

There are some areas of the site in the north-eastern part of Stage 1 and across Stage 2 where only limited investigations were able to be carried out due to access constraints (e.g. tracks not passable for machinery or dense vegetation). Further investigations will be required within these areas as part of inputs for detailed design of the development.

Geotechnical design inputs will be required for specific design of the stability enhancement measures, mechanically stabilised e[redacted] retaining walls etc. Further investigation may also be required to enable specific designs to be completed.

7.0 Summary

Based on the field investigation findi[redacted]chnical assessments, we consider that the proposed development is suitable fo[redacted]ct to the recommendations outlined in this report. These are summarised below.

- As a result of our qualitative and quantitative stability assessments, we consider that adequate FoS against instability should be available across the site with construction of stability enhancement me[redacted]ssed in Section 5.1.
- While the proposed develo[redacted]ot meet the permitted E7 AUP standards with respect to groundwater drawdown, the extent of groundwater drawdown is limited and there are no structures within the zone of influence of the groundwater drawdown.
- Shallow foundations are considered to be appropriate for NZS 3604 type residential dwellings up to three levels where setback from the steeper slopes and clear of stability enhancement measures.
- The soils encountered on-site have been assessed as likely ranging from Class M (Moderate) to E (Extreme), with respect to AS 2870:1996. Further expansive soils laboratory testing is recommended at the geotechnical completion reporting stage to delineate areas of differing expansive soil classification.
- Earthworks should be undertaken in accordance with NZS 4431. Preliminary soil compaction criteria and earthworks recommendations are provided.
- Geotechnical observations and testing will be required during site development earthworks and service line installation.
- Further geotechnical input will be required for the detailed design of the stability enhancement measures. Due to access constraints, further geotechnical investigations are also required at some locations to support detailed design of the stability enhancements.

8.0 Limitation

This report has been prepared solely for the benefit of Vineway Ltd as our client with respect to the brief and Auckland Council in processing the consent. The reliance by other parties on the information or opinions contained in the report shall, without our prior review and agreement in writing, be at such parties' sole risk.

Recommendations and opinions in this report are based on data from limited test positions. The nature and continuity of subsoil conditions away from the test positions are inferred, and it must be appreciated that actual conditions could vary considerably from the assumed model.

[redacted]

[redacted]

[redacted]

[redacted]

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