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Beachlands South Fast-Track Application

Geotechnical Report

Prepared for

Beachlands South Limited Partnership

Prepared by

Tonkin & Taylor Ltd

Date

June 2026

Job Number

1014358.3200 v1.2

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Document control

Title: Beachlands South Fast-Track Application					
Date	Version	Description	Prepared by:	Reviewed by:	Authorised by:
8/05/2026	1	DRAFT for Client review	Scott Honiss James Lee	Jacob Simpson Mark Thomas	Mark Thomas
15/06/2026	2	Earthworks update	James Lee	Jacob Simpson Mark Thomas	Mark Thomas
19/06/2026	2.1	Minor updates		James Lee	Mark Thomas
19/06/2026	2.2	Minor updates			Mark Thomas

Distribution:

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Statement of qualifications and experience

My name is Mark Robertshaw Thomas. I am a Chartered Professional Geotechnical Engineer and Senior Principal at Tonkin & Taylor Limited (T+T), an environmental and engineering consultancy firm.

I have a BE (Hons) from the University of Auckland. I have 25 years post-graduate experience in geotechnical and civil engineering and am a member of Engineering New Zealand and the Geotechnical Society of New Zealand. Recent projects and services relevant to this engagement include:

- i Geotechnical Design Manager for the Piritahi Auckland Housing Programme (more recently rebranded as “Lead Alliance”), covering residential redevelopment of large areas within Auckland City;
- ii Lead Geotechnical Engineer for Commercial Bay and interfacing with the CRL tunnels that pass through it;
- iii Geotechnical Project Director for large central urban developments (the proposed Mission Bay and Devonport redevelopments);
- iv Geotechnical Expert and Project Director for several Watercare wastewater upgrades (Motions CC2, Kelmarna – Grey Lynn, Herne Bay, Avondale CC6, Hingaia);
- v Geotechnical Project Director for the Hunua Quarry Expansion Fast Track application;
- vi Geotechnical Project Director for the Beachlands South subdivision;
- vii Geotechnical Design Manager for Waterview Connection surface works;
- viii Project Director for 29 properties within the Kainga Ora Wellington Region Housing programme;
- ix Design Manager and later Project Director for remediation of the large Rawene landslide in Birkenhead, Auckland;
- x Project Director, civil and geotechnical works for Stages 1 and 2 of the St Ignatius of Loyola Catholic College in Drury; and
- xi Geotechnical Project Director for Parklands Estate Area 4 subdivision, Napier.

I confirm that I have read the Code of Conduct for expert witnesses contained in the Environment Court of New Zealand Practice Note 2023 and that I have complied with it when preparing this report. Other than when I state I am relying on the advice of another person, this report is within my area of expertise. I have not omitted to consider material facts known to me that might alter or detract from the opinions that I express.

1 Executive Summary

This Geotechnical Report has been prepared to assess the substantive application submitted by Beachlands South Limited Partnership under the Fast-Track Approvals Act 2024 ('FTAA'). The Beachlands South Project ("Project") includes the properties at 110 Jack Lachlan Drive and 620, 712, 758 and 770 Whitford-Maraetai Road, Beachlands.

The Project will provide for the development of the remaining live-zoned land at Beachlands South (the "Site"), which involves converting part of the golf course at 110 Jack Lachlan Drive as well as the use of a portion of Future Urban Zone ('FUZ') land for wastewater management purposes.

The main conclusions of this report are listed below:

Ground and groundwater conditions	The Site is underlain by typical Auckland East Coast Bays Formation terrain, which has already been modified as part of development of the existing golf course. The proposed development is mainly within the flatter land of the Project area and is set back from the coastal cliffs and gullies. Perched groundwater was encountered between 0.1 m and 5.0 m below current ground level. The underlying regional groundwater level is deeper, at the level of the base of the gullies and the surrounding coastline. Further groundwater measurement and monitoring will be carried out for specific excavation, slope stability and liquefaction analysis as detailed in this report. Refer to Section 3 of this report for more detail.
Slope stability (including landslide hazard)	Landslide hazards have been assessed through review of existing slopes, proposed earthworks design (prepared by Harrison Grierson Consultants Limited), the maps prepared as part of Plan Change 120 ("PC120") to the Auckland Unitary Plan Operative in Part ("AUP:OP"), and analysis of critical and representative slope stability models. The overall landslide risk in terms of the National Policy Statement for Natural Hazards ("NPS-NH") and PC120 is considered low (as discussed in Section 4.9.2 of this report). Most of the proposed slopes do not require specific engineering stabilisation measures in order to meet the stability requirements of the Auckland Code of Practice for Land Development and Subdivision (Ref. [5]). Preliminary analyses indicate that engineering stabilisation measures will be required for parts of the southern coastal cliffs, to achieve acceptable factors of safety against instability. We have recommended a consent condition for this area to achieve the required levels of stability (as detailed in Section 7.1 of this report). Refer to Section 4.3 and Section 4.9.2 of this report for more detail.
Liquefaction	In the context of the AUP:OP Chapter E36 (Natural Hazards and Flooding, as amended by PC120) and NPS-NH, liquefaction is not expected to present a significant constraint to the proposed development. The liquefaction risk is considered low and can be appropriately managed through detailed design and standard geotechnical construction practices. Refer to Section 4.2.4 and Section 4.9.3 of this report for more detail.
Active faults	No active fault traces are mapped within the Site; this risk is low. In the context of the AUP:OP Chapter E36 (Natural Hazards and Flooding, as amended by PC120) and NPS-NH, the risk of surface fault rupture affecting the proposed development is considered low and not a constraint to the proposed subdivision development. Refer to Section 4.9.4 of this report for more detail.
Other natural hazards	Coastal erosion, coastal inundation and tsunami hazards are addressed separately in the T+T Coastal Hazards report (Ref. [23]). Catchment flooding is addressed in a separate report by Harrison Grierson Consultants Limited.

Groundwater impact	The proposed earthworks are assessed to meet the Permitted Activity criteria under AUP:OP E7.6.1.10; specifically: 1 A conservative analysis of groundwater drawdown shows that the natural groundwater level will not be reduced by more than 2m on the boundary of any adjoining site. 2 Water take or diversion are not expected within a Natural Stream Management Area Overlay, or Wetland Management Area Overlay. 3 The earthworks cuts are not expected to encounter the general regional groundwater table, which is at/near the level of the surrounding gullies and coastline. 4 Drainage measures are not expected to extend deeper than 2 m below natural groundwater level (subsoil drains are only anticipated to drain perched groundwater lenses). Architectural concepts for several of the buildings proposed for the Site include basement car parks that extend up to one level below ground. Some of the basements will encounter perched groundwater, though none are likely to encounter the underlying regional groundwater table. Conditions of consent are recommended to ensure that appropriate geotechnical methodologies are employed in the design and construction of the basements (in terms of AUP:OP E7.6.1.10 for diversion of groundwater).
Earthworks (including historic fill)	Proposed earthworks (designed and prepared by Harrison Grierson Consultants Limited) are expected to comprise cuts of up to approximately 5 m (locally up to 9 m for ponds) and fills of up to approximately 9 m to achieve design levels across the Site. Consolidation settlements beneath the weight of earthworks fills will occur relatively rapidly, and residual settlements are expected to be negligible. The in-situ soils will generally be suitable for use as site-won fill, during summer months. Care and conditioning of fill is required when handling soils, due to their sensitivity to moisture. Subsoil drainage may be required at the invert level of any gullies or wet ground (localised seepages) encountered during earthworks. Refer to Section 4.5 and Section 5.2 of this report for more detail.
Future building foundations (including expansive soils)	The developed Site will generally provide Good Ground conditions (in terms of NZS 3604:2011 / NZS 4229:2013) with the following exceptions: 1 Site testing shows that the soils across the Site are expansive, and lot-specific testing should be carried out to confirm whether foundations should be designed for Class M or Class H in terms of AS 2870:2011. 2 Liquefaction risk at the proposed CM4 (Terrace Housing) and CM5 (Hotel) will require further investigation and assessment at building consent stage. Refer to Sections 4.2.4 and 5.3 of this report for more detail.
Future basement structures and retaining walls	Several of the proposed buildings within the Site include basement car parks extending up to one level below ground. It is likely that some of the basements will encounter perched groundwater. Resource consent conditions are recommended to ensure that appropriate geotechnical methodologies are employed in the design and construction of retaining walls and basements. Refer to Section 5.4 and Section 5.5 of this report for more detail.
Recommended conditions of consent and further work	Consent conditions are recommended to verify ground conditions during construction, and ensure that appropriate geotechnical methodologies are employed in the design and construction of slope stabilisation measures, retaining walls, building basements, and works within the FUZ. Recommended consent conditions are listed in Section 7 of this report.

2 Introduction

2.1 Report purpose

This Geotechnical Report has been prepared to assess the substantive application submitted by BSLP under the Fast-Track Approvals Act 2024 ('FTAA'). The Project name is 'Beachlands South' and is included in Schedule 2 of the FTAA as a Listed Project with significant regional or national benefits. The Project is located across approximately 159.58 hectares of live zoned land and approximately 108.6 hectares of Future Urban zoned land at 110 Jack Lachlan Drive and 620, 712, 758 and 770 Whitford-Maraetai Road, Beachlands (the "project area", Ref. [2]).

2.2 Project description

2.2.1 Live Zone

In summary, the Project will provide for the development of the live-zoned land at Beachlands South (as shown in Figure 2.1) into a comprehensively planned and public transport focussed community to achieve a well-functioning urban environment comprising residential dwellings, a hotel and conference venue, community and commercial activities, protected ecological restoration areas, walking and cycling networks, and community facilities, and enable the potential development of two schools, as well as all associated earthworks and all associated infrastructure including transport facilities, intersection upgrades and roading, telecommunications, power, and water supply, stormwater, and wastewater infrastructure (including any treatment or storage facilities, drainage, pipes, pump stations, outfalls and discharges). A portion of FUZ land will be used for wastewater management purposes.

Key elements of the Project include:

- 3 Residential activities comprising up to approximately 2,000 dwellings in a variety of housing types, densities and vacant sites for future residential development;
- 4 Establishment of a commercial local centre supported by employment opportunities within and around the periphery of the local centre and on the Whitford-Maraetai Road frontage comprising approximately 27,000 m² of commercial GFA and 30,000 m² of light industrial GFA;
- 5 Education facilities comprising the development of both primary and secondary schools;
- 6 Establishment of an Ecological Protected Area Network (EPAN) comprising approximately 88 hectares of land within the live zoned area being protected and approximately 55 hectares of land being replanted, restored, maintained and enhanced including preservation of streams and their margins;
- 7 A series of open spaces including a suburban park, neighbourhood parks, the Fairway Reserve, golf course, esplanade reserve and riparian margins;
- 8 Provision of public access to and along the coastal marine area and the creation of a quality coastal pathway;
- 9 Public transport enhancements and integrations including expanded ferry passenger capacity;
- 10 Staged road upgrades including the provision of pedestrian and cycling facilities along Jack Lachlan Drive, the upgrade of Whitford roundabout, a series of intersection upgrades; and
- 11 Establishment of infrastructure to service the project including wastewater, water supply, stormwater, telecommunications and power.

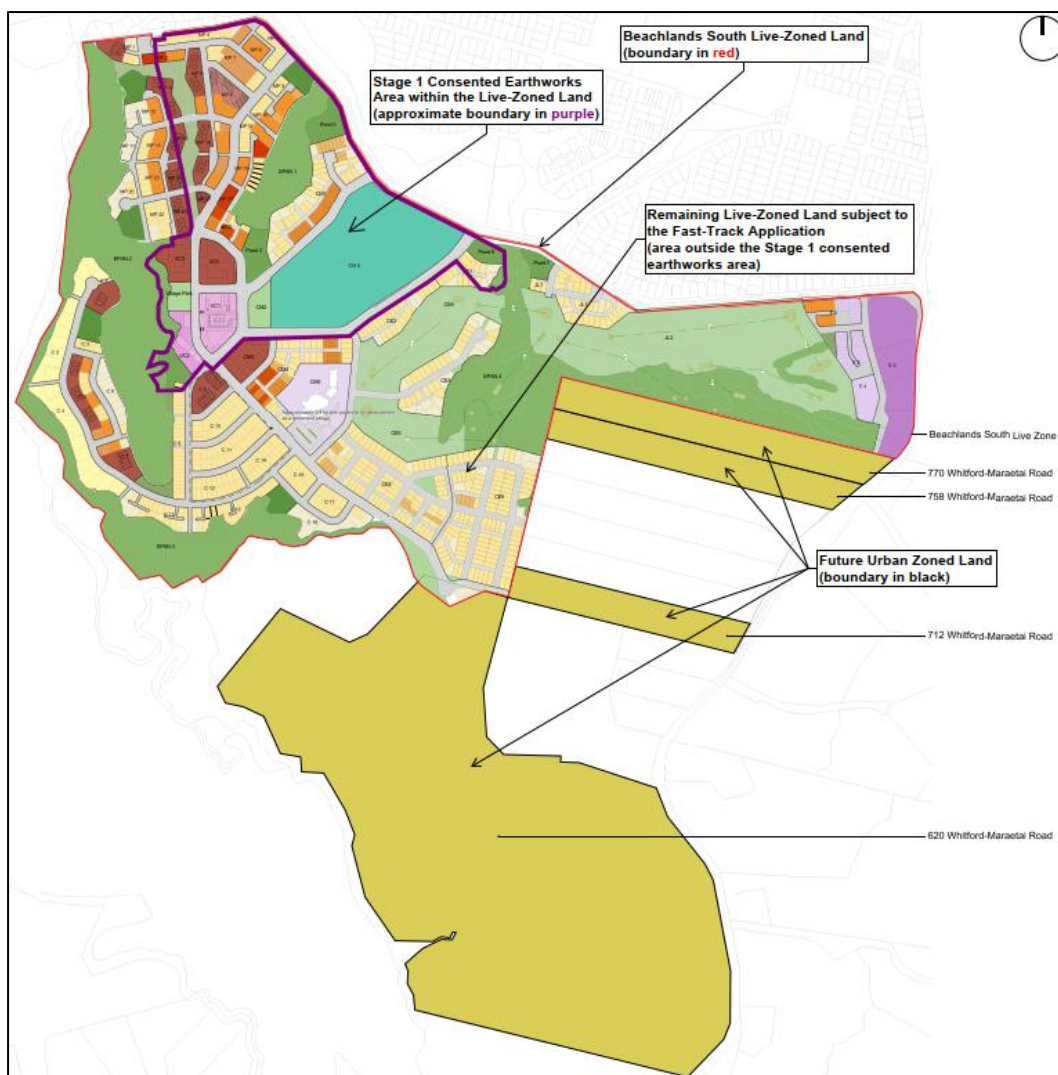


Figure 2.1: Remaining live-zoned land subject to this Fast-Track Application

2.2.2 Future Urban Zone

The properties at 620, 712, 758 and 770 Whitford-Maraetai Road (and a small part of the property at 110 Jack Lachlan Drive) are zoned FUZ. Limited works are proposed within the FUZ:

- 1 620 Whitford-Maraetai Road will contain some of the wastewater discharge via drip feed irrigation.
- 2 712 Whitford-Maraetai Road will have an access road located on it to serve the development.
- 3 770 and 758 Whitford-Maraetai Road are proposed to be used for potential additional golf holes and an access way to the light industrial / work live units on the land fronting Whitford-Maraetai Road.

Commentary on geotechnical considerations for the FUZ are provided in section 5.6 of this report.

2.3 Scope of work

The objective of this Geotechnical Report is to describe the geotechnical investigation completed within the live zoned Site, the ground conditions encountered, identify and quantify geotechnical risks associated with development of the Site, and provide geotechnical recommendations relating to site preparation, temporary works, foundation construction and site / subsoil classification.

This report addresses the following scope of work:

- 1 Desktop review of previous investigations and reporting undertaken at the Site;
- 2 Site-specific geotechnical investigations and laboratory testing (Section 3.6 and Section 3.7);
- 3 Develop geological ground models representative of the Site (Section 3.8);
- 4 Assess site-specific geotechnical hazards (Section 4), including an assessment in accordance with the AUP:OP (Ref. [3]) Chapter E36 (Natural Hazards and Flooding, as amended by PC120) and the NPS-NH (Ref. [4]); and
- 5 Provide site-specific geotechnical recommendations (Section 5) in accordance with the Auckland Code of Practice for Land Development and Subdivision Section 2 (Earthworks and Geotechnical Requirements) (Ref. [5]).

The 2022 Structure Plan Geotechnical Report (Ref. [8]) describes ground conditions in the FUZ. While the FUZ land forms part of the wider Project Area, only limited development is currently proposed within these properties. Accordingly, no geotechnical investigation, analysis, or detailed assessment has been undertaken for the FUZ land. Commentary on geotechnical considerations for the FUZ are provided in section 5.6 of this report.

3 Ground conditions

3.1 Published geology

The published geological map (Ref. [6], see Figure 3.1) indicates that the proposed Beachlands South development area (including both the Site and FUZ land) is primarily underlain by deposits of the East Coast Bays Formation (“ECBF”). This formation typically comprises interbedded sandstone, siltstone, and mudstone, with variable volcanic content and occasional volcanoclastic grits.

Alluvial deposits of the Tauranga Group are mapped along the western coastal margin of the Site (i.e. between the shoreline and cliff face) and are also likely present within the base of incised stream channels. These deposits typically comprise younger, unconsolidated sediments including silts, sands, and minor gravels.

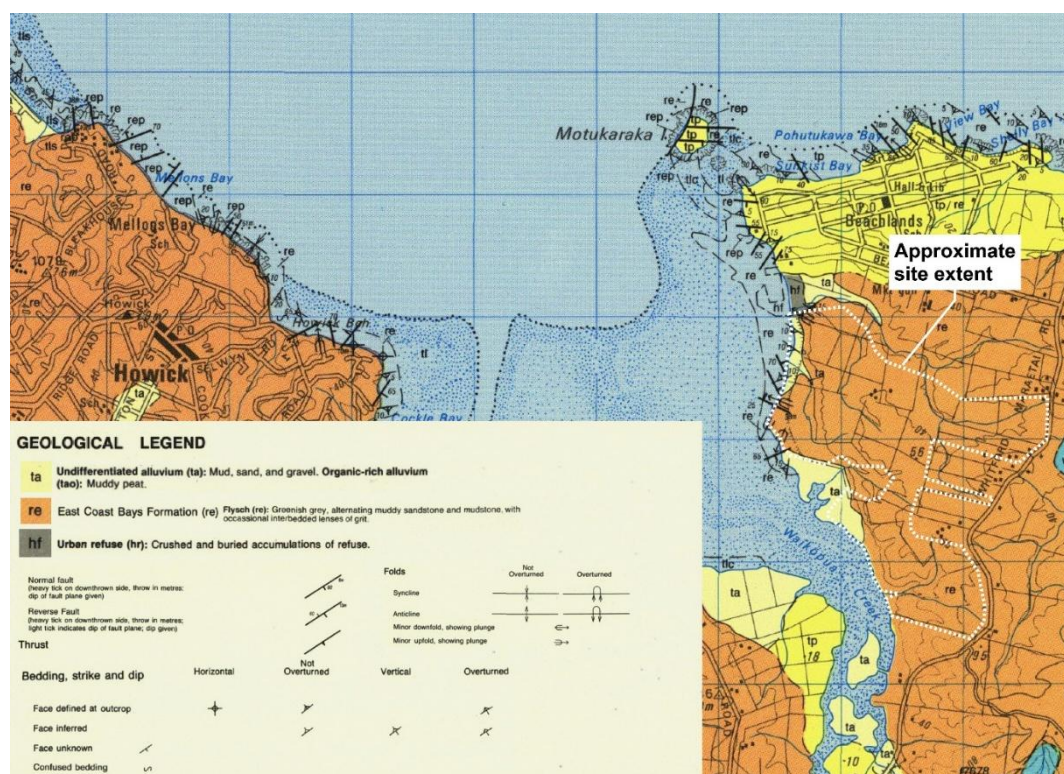


Figure 3.1: Published geology and site setting of the Site and FUZ land (approximate boundary in white)

3.2 Site geomorphology

The structural geology of the ECBF rock mass at the Site is well exposed along the western coastal cliffs, where bedding dips vary significantly (approximately 10° to 70°) due to localised folding. Along the northern coastal margin, the fold axis generally plunges toward the northwest, transitioning to a southwest plunge in the vicinity of Waikopua Creek. The variability in bedding orientation reflects localised folding and has potential implications for slope behaviour, particularly where bedding dips daylight toward slope faces.

The bedrock is further structured by several faults interpreted from regional geological mapping, including NW–SE trending normal faults, an approximately E–W trending reverse fault, and a southeast-dipping normal fault. These features reflect the broader structural complexity of the ECBF in the area and contribute to the observed geomorphological patterns across the Site.

The coastal margin is characterised by a series of terraced landforms, which aerial imagery and site observations indicate are relict features formed by historical sea-level fluctuations, potentially

influenced by underlying geological structure. While the western coastal slopes are generally stable at a global scale, a notable scallop-shaped landslide feature, is present. This feature is considered to have resulted from weakened bedding of the ECBF from faulting accelerated by localised erosion and drainage processes.

Further inland, the Site is intersected by several deeply incised gullies, with slope gradients locally steeper than 1V:3H. The flanks of these gullies exhibit geomorphic indicators consistent with ongoing surficial soil creep, including hummocky ground and leaning vegetation. Localised areas of more recent instability have also been observed, including a shallow landslide adjacent to Jack Lachlan Drive (located outside the current Fast-Track application earthworks area, remaining as a golf course fairway), which has retrogressed toward the crest and impacted the adjacent roadway. It is noted that the steeper gullied areas (where instability is most evident) are largely located within the identified EPAN or open space areas (where earthworks are not proposed), limiting their interaction with proposed development. In contrast, the broader interfluvial and gently sloping areas of the Site are comparatively stable.

Finally, much of the interior Site has been modified by historical earthworks associated with the existing golf course. These works have involved localised cut-and-fill operations to form fairways, greens, tee boxes, and artificial water features.

Overall, slope instability within the Site is primarily associated with steep coastal margins and incised gully systems, while the majority of the developable landforms are expected to exhibit relatively stable ground conditions.

3.3 Historical aerial photography

Aerial imagery from 1939 to 2025 (presented in Appendix A and summarised in Table 3-1) shows progressive land use changes at the Site, with major earthworks for the Formosa Golf Resort visible in 1996. Gully infill and cut/fill areas are evident (as shown in Figure 3.2). The extent and preparation of historical fill is undocumented, but boreholes (2016 by Lander Geotechnical Consultants, Lander (Ref. [7]), 2021 by T+T (Ref. [8]), and 2025 by T+T (Ref. [9] and Ref. [10])) encountered fill throughout the Site, comprising very stiff to hard silt/clay, indicating good compaction. Given the fill's age, we consider that the risk of significant settlement associated with self-weight of the historic earthworks to be relatively low. Settlement assessments for the Project are presented in Section 4.6 of this report.

Table 3-1: Observed land use visible in historical aerial photos

Year of Imagery	Changes in Land Use
1939	Beachlands is lightly populated. There is very light farming activity in the live-zoned area.
1955	A few houses are visible in the Formosa site area, and light farming activity.
1961	No change from 1955.
1972	More intensive farming, more small structures, and valleys have been bridged by either structures or infilling.
1980	More structures, but generally little change since the 1972 photo.
1996	Earthworks for construction of Formosa Golf Resort. Pine Harbour marina appears in the photo.
2001	Formosa Golf Resort in operation.
2017	Housing developments beginning in the property north of Formosa Golf Resort.
2025	No major developments apart from significant tree removal within gulley in northeast of the live-zoned area.

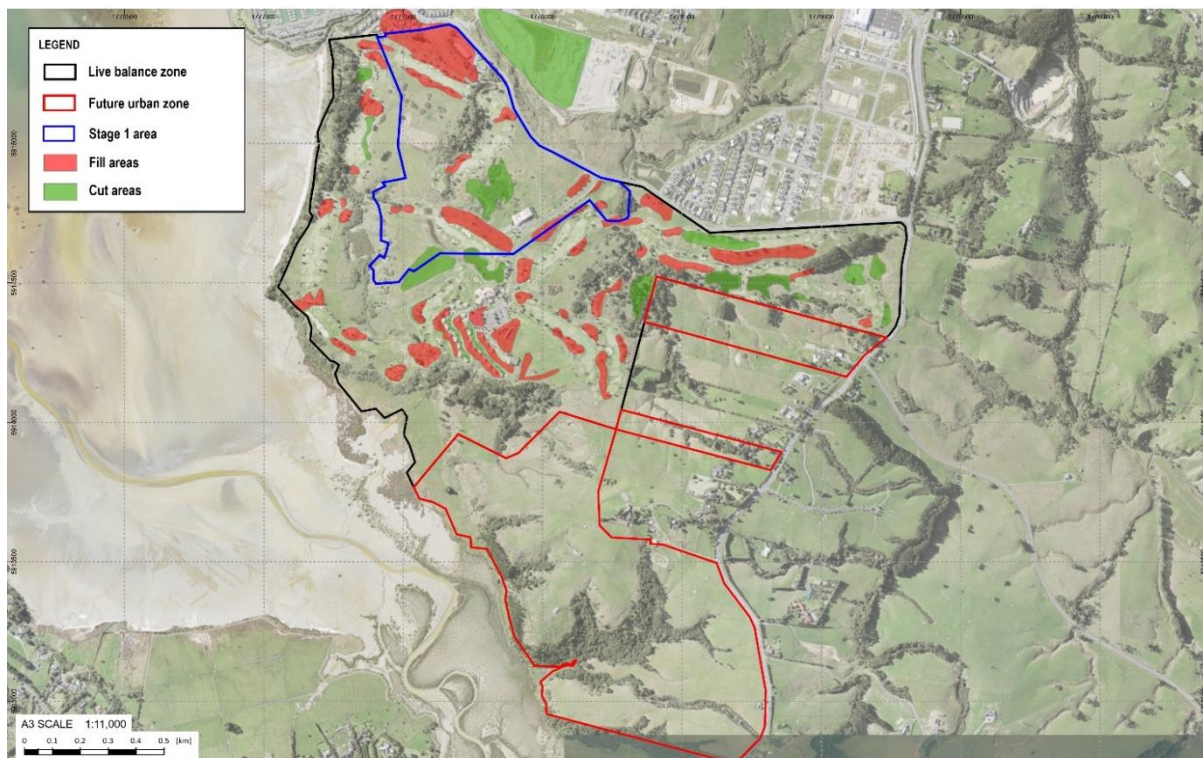


Figure 3.2: Areas of historic cut and fill across the Formosa Golf Resort, inferred from a review of historical aerial photos

3.4 Historical geotechnical investigations

Previous geotechnical investigations were carried out by Lander (Ref. [7]) and T+T (Ref. [8]) across the Site and the FUZ. The locations of these historic investigations within the Site are shown in 1014358.3200–Figure 1 in Appendix A, with logs included in Appendix B.

A summary of the historical geotechnical investigations is presented in Table 3-2 and has been used to supplement the interpretation of the Site's subsurface conditions.

Historical investigations within the FUZ are not covered by this report. The 2022 Structure Plan Geotechnical Report (Ref. [8]) describes ground conditions in the FUZ.

3.5 T+T 2025 Stage 1 geotechnical investigations

Targeted investigations were undertaken within the Stage 1 consented earthworks area in May 2025 (the area shown in Figure 2.1), and comprised:

- 1 15 No. Cone Penetrometer Tests (CPTs) with pore pressure measurement.
- 2 30 No. Hand Augered Boreholes (HAs).
- 3 2 No. Hand Dug Trial Pits (TPs).

The locations of these recent investigations are shown in 1014358.3200–Figure 1 in Appendix A, with logs included in the T+T Stage 1 Geotechnical Report (Ref. [9]) and the T+T Stage 1 Geotechnical Letter (Ref. [10]). This data (not reproduced in this report) has been used to supplement the interpretation of the Site's subsurface conditions.

Table 3-2: Summary of historical geotechnical investigation details

Investigation ID	Location (NZTM2000)		Ground Surface Elevation (NZVD2016, mRL)	Depth Achieved (m bcgl, below current ground level)
	Easting (m)	Northing (m)		
TT_HA108 [2021]	1777242	5914649	19.0	4.2
TT_HA109 [2021]	1777234	5914403	23.0	4.2
TT_HA110 [2021]	1777561	5914246	27.0	4.2
TT_HA111 [2021]	1778205	5914063	53.0	4.2
TT_HA115 [2021]	1779098	5914594	37.0	2.5
TT_HA116 [2021]	1779222	5914313	52.0	4.2
Lander_BH01 [2016]	Not recorded ⁽¹⁾		23.0 ⁽²⁾	4.0
Lander_BH02 [2016]	Not recorded ⁽¹⁾		27.0 ⁽²⁾	4.0
Lander_BH03 [2016]	Not recorded ⁽¹⁾		22.0 ⁽²⁾	4.0
Lander_BH04 [2016]	Not recorded ⁽¹⁾		33.0 ⁽²⁾	4.0
Lander_BH05 [2016]	Not recorded ⁽¹⁾		27.0 ⁽²⁾	4.0
Lander_BH06 [2016]	Not recorded ⁽¹⁾		44.0 ⁽²⁾	4.0
Lander_BH07 [2016]	Not recorded ⁽¹⁾		34.0 ⁽²⁾	4.0
Lander_BH08 [2016]	Not recorded ⁽¹⁾		48.0 ⁽²⁾	4.0
Lander_BH09 [2016]	Not recorded ⁽¹⁾		38.0 ⁽²⁾	4.0
Lander_BH10 [2016]	Not recorded ⁽¹⁾		30.0 ⁽²⁾	4.0
Lander_BH11 [2016]	Not recorded ⁽¹⁾		39.0 ⁽²⁾	4.0
Lander_BH12 [2016]	Not recorded ⁽¹⁾		45.0 ⁽²⁾	4.0
PDP_BH23094 [2008]	1777414	5915392	19.6	179.5

Notes:

(1) Locations shown in 1014358.3200–Figure 1 (Appendix A) are approximated based on Lander’s site plan (Ref. [7]).

(2) Ground surface elevations are estimated from available contour information and the estimated investigation location.

3.6 T+T 2026 geotechnical investigations

3.6.1 General

Targeted investigations were undertaken within the Site from 14 to 23 April 2026, and comprised:

- 1 23 No. Cone Penetrometer Tests (CPTs) with pore pressure measurement.
- 2 35 No. Hand Augered Boreholes (HAs).

Investigation locations were selected based on proposed cut/fill zones and are shown in 1014358.3200–Figure 1 (Appendix A).

3.6.2 Cone Penetration Tests (CPTs)

The CPTs were undertaken from 14 to 23 April 2026 using an anchored CPT rig, supplied and operated by Geotechnics Limited (Geotechnics). The CPTs were advanced from current ground level to a target depth of approximately 20m or to “refusal”, which occurs due to either:

- 1 The CPT cone terminating on or within a hard impenetrable stratum (e.g. rock or a dense sand layer);

- 2 The skin friction of the soil on the side of the CPT probe exceeding the operating level;
- 3 Pull out of the anchors that restrain the CPT rig; or
- 4 Excessive pore pressure exceeding the operating level.

CPT logs are presented in Appendix C, with details summarised in Table 3-3.

Table 3-3: Summary of CPT details

CPT ID	Location (NZTM2000)		Ground Surface Elevation (NZVD2016, mRL)	Depth Achieved (m bcgl, below current ground level)	Reason for termination
	Easting (m)	Northing (m)			
CPT-001	1777332.7098	5915340.8281	16.5	14.81	Skin friction
CPT-002	1777440.3382	5915189.2193	25.6	20	Target depth
CPT-003	1777353.3411	5915115.4772	20.5	15.63	Pore pressure
CPT-004	1777467.3274	5915009.1822	27.9	20	Target depth
CPT-005	1777362.668	5914932.0279	18.4	12.98	Skin Friction
CPT-006	1777257.2219	5914720.2077	16.8	19.45	Pore Pressure
CPT-007	1777210.5352	5914519.1715	23.5	20	Target depth
CPT-008	1777259.5528	5914422.2874	20.9	14.89	Anchor failure
CPT-009	1777434.7952	5914208.5164	17.9	19.25	Skin Friction
CPT-010	1777625.9497	5914280.8502	32.8	9.81	Pore pressure
CPT-011	1777553.6915	5914516.6546	30.4	18.82	Skin Friction
CPT-012	1777771.7935	5914204.5963	36.6	18.81	Anchor Failure
CPT-013	1777810.7553	5914414.1595	42.1	7.17	Pore pressure
CPT-014	1777822.5907	5914597.4332	36.6	20	Target depth
CPT-015	1777974.2893	5914661.5504	36.8	13.46	Pore pressure
CPT-016	1778136.6502	5914559.231	36.8	9.02	Pore pressure
CPT-017	1778034.8719	5914217.7325	47.2	13.26	Pore pressure
CPT-018	1778190.6469	5914280.3644	50.7	13.11	Pore pressure
CPT-019	1778083.3638	5914041.2268	47.1	15.63	Pore pressure
CPT-020	1778372.1354	5914762.6508	24.4	5.68	Skin Friction
CPT-021	1779105.8324	5914636.5362	46.4	9.94	Skin Friction
CPT-022	1779249.285	5914558.8846	56.8	8.78	Skin Friction
CPT-023	1779223.953	5914371.876	50.05	12.75	Point load

3.6.3 Hand augered boreholes (HAs)

The HAs were undertaken from 14 to 23 April 2026 by Geotechnics engineering geologists and geotechnical technicians. In-situ undrained shear strength testing was undertaken at 0.3 m intervals throughout the soil horizon using a handheld shear vane. The recovered drill cores were photographed and logged in general accordance with NZGS 'Field Description of Soil and Rock' guidelines. HA logs and core photographs are presented in Appendix C, with details summarised in Table 3-4.

Soil samples for laboratory testing were retrieved from recovered core at targeted depths within specific HAs. Further details are outlined in Section 3.7.

Table 3-4: Summary of HA details

HA ID	Location (NZTM2000)		Ground Surface Elevation (NZVD2016, mRL)	Depth Achieved (m bcgl, below current ground level)	Piezometer Install
	Easting (m)	Northing (m)			
BH001 ⁽¹⁾	1777427	5915253	25	5.0	Yes
BH002 ⁽¹⁾	1777355	5915238	16.5	5.0	Yes
BH003 ⁽¹⁾	1777459	5915101	27	5.0	Yes
BH004	1777384	5915005	23.5	5.0	No
BH005 ⁽¹⁾	1777478	5914920	28	3.6 ⁽²⁾	Yes
BH006 ⁽¹⁾	1777225	5914799	21	5.0	Yes
BH007 ⁽¹⁾	1777243	5914622	21	5.0	Yes
BH008 ⁽¹⁾	1777128	5914551	21	5.0	Yes
BH009 ⁽¹⁾	1777284	5914502	19	4.0 ⁽²⁾	Yes
BH010	1777201	5914400	23	4.0 ⁽²⁾	No
BH011 ⁽¹⁾	1777305	5914329	21.5	4.3 ⁽²⁾	Yes
BH012	1777355	5914218	19	5.0	No
BH013	1777498	5914317	22	5.0	No
BH014	1777578	5914368	31	5.0	No
BH015 ⁽¹⁾	1777622	5914282	33	5.0	Yes
BH016	1777626	5914501	36	5.0	No
BH017 ⁽¹⁾	1777634	5914609	33	3.7	Yes
BH018 ⁽¹⁾	1777709	5914498	39	5.0	Yes
BH019	1777724	5914308	37.5	5.0	No
BH020 ⁽¹⁾	1777774	5914187	35	5.0	Yes
BH021 ⁽¹⁾	1777811	5914404	41.5	5.0	Yes
BH022 ⁽¹⁾	1777952	5914657	37	5.0	Yes
BH023	1777912	5914303	46.5	5.0	No
BH024	1778059	5914476	43	5.0	No
BH025 ⁽¹⁾	1778222	5914650	25	4.1	Yes
BH026 ⁽¹⁾	1778061	5914139	47	5.0	Yes
BH027	1778103	5914240	50	5.0	No
BH028 ⁽¹⁾	1778122	5914274	50	5.0	Yes
BH029	1778045	5914197	47	5.0	No
BH030 ⁽¹⁾	1778369	5914788	27.5	5.0	Yes
BH031	1778493	5914706	32	5.0	No
BH032 ⁽¹⁾	1779230	5914676	53	5.0	Yes
BH033	1779219	5914571	51	2.3 ⁽²⁾	No
BH034 ⁽¹⁾	1779145	5914509	48	5.0	Yes
BH035	1779257	5914458	55	5.0	No

Notes:

- (1) Refer to Section 3.6.4.1 for piezometer installation details.
(2) Scala penetrometer testing undertaken in base of hole to 5m or refusal.

3.6.4 Groundwater

3.6.4.1 Piezometer details

Standpipe piezometers were installed by Geotechnics in 21 boreholes upon completion, with level-loggers subsequently installed on 21 and 22 May 2026 to continuously monitor groundwater levels at 15-minute intervals.

Piezometer installation records are presented in Appendix C, with details summarised in Table 3-5.

Table 3-5: Standpipe piezometer details

Investigation ID	Piezometer Installation Screen Depth (m bcgl, below current ground level)	Level Logger Type	Geological Unit Over Screened Depth
BH001	4.0 – 5.0	Solinst M20 Logger	Residual ECBF
BH002	1.5 – 3.5	Solinst M10 Logger	Residual ECBF
BH003	3.0 – 5.0	Solinst M20 Logger	Residual ECBF
BH005	1.5 – 3.5	Solinst M10 Logger	Residual ECBF
BH006	3.0 – 5.0	Solinst M20 Logger	Residual ECBF
BH007	3.0 – 5.0	Solinst M30 Logger	Residual ECBF
BH008	1.0 – 3.0	Solinst M20 Logger	Residual ECBF
BH009	1.0 – 3.0	Solinst M20 Logger	Residual ECBF
BH011	2.3 – 4.3	Solinst M10 Logger	Residual ECBF
BH015	2.0 – 4.0	Solinst M10 Logger	Residual ECBF
BH017	1.0 – 3.0	Solinst M10 Logger	Residual ECBF
BH018	3.0 – 5.0	Solinst M30 Logger	Residual ECBF
BH020	2.0 – 4.0	Solinst M30 Logger	Residual ECBF
BH021	3.0 – 5.0	Solinst M30 Logger	Residual ECBF
BH022	1.0 – 3.0	Solinst M30 Logger	Residual ECBF
BH025	1.0 – 3.0	Solinst M20 Logger	Residual ECBF
BH026	2.5 – 4.5	Solinst M10 Logger	Residual ECBF
BH028	1.0 – 3.0	Solinst M10 Logger	Residual ECBF
BH030	3.0 – 5.0	Solinst M10 Logger	Residual ECBF
BH032	3.0 – 5.0	Solinst M20 Logger	Residual ECBF
BH034	1.0 – 3.0	Solinst M10 Logger	Residual ECBF

3.6.4.2 Groundwater levels

Groundwater was measured during the investigations in April 2026, and again during the installation of the level-loggers in May 2026. The measurements presented in Table 3-6 below show the manual dips undertaken during these periods, with ongoing monitoring currently being obtained from level-loggers installed within the piezometers. The recorded groundwater data is presented in Appendix C. Piezometer / borehole locations are shown in 1014358.3200–Figure 1 (Appendix A).

Table 3-6: recorded groundwater levels

Investigation ID ⁽¹⁾	Groundwater depth (m bcgl, below current ground level)		
	During investigation – April 2026 ⁽²⁾	Post investigation – May 2026 ⁽³⁾	Notes
BH001	3.20	Dry	-
BH002	3.00	2.30	-
BH003	3.40	2.00	Inflow recorded at 0.80 m and at 3.40
BH004	2.30	-	-
BH005	-	0.97	Inflow recorded at 0.50
BH006	4.87	Dry	Groundwater not encountered
BH007	1.00	2.30	-
BH008	0.85	1.00	Inflow recorded at 0.90
BH009	0.27	0.28	Investigation undertaken in area of wet surficial soil
BH010	3.30	-	Inflow recorded at 2.80, standing water at 3.30 one hour after completion.
BH011	1.50	Dry	Inflow recorded at 2.00
BH012	0.80	-	Inflow recorded at 1.80
BH013	3.80	-	-
BH014	2.40	-	-
BH015	Dry on completion	4.00	Minor inflow recorded at 0.90
BH016	1.45	-	Minor Inflow recorded at 1.80
BH017	-	2.40	Inflow recorded at 3.30
BH018	3.80	2.70	Inflow recorded at 3.00
BH019	1.00	-	-
BH020	-	2.60	Inflow recorded at 2.50
BH021	4.20	3.00	-
BH022	1.20	1.52	Inflow recorded at 1.80
BH023	0.10	-	Inflow recorded at 0.90, potential surface flow into hole occurred during drilling.
BH024	0.20	-	Inflow recorded at 2.80
BH025	0.15	0.87	Inflow recorded at 0.80
BH026	1.80	1.80	Inflow recorded at 1.80
BH027	Dry on completion	-	Groundwater not encountered
BH028	1.30	1.90	Inflow recorded at 2.00
BH029	0.20	-	-
BH030	4.60	4.40	Inflow recorded at 3.90
BH031	2.00	-	Inflow recorded at 3.70
BH032	Dry on completion	Dry	Inflow recorded at 4.60
BH033	Dry on completion	-	Groundwater not encountered
BH034	5.00	4.22	Inflow recorded at 4.00

Investigation ID ⁽¹⁾	Groundwater depth (m bcgl, below current ground level)		
	During investigation – April 2026 ⁽²⁾	Post investigation – May 2026 ⁽³⁾	Notes
BH035	4.20	-	Inflow recorded at 1.40

Notes:

- (1) Investigation locations are shown in 1014358.3200–Figure 1 (Appendix A).
 (2) Measurements obtained from dip meter during investigation phase.
 (3) Measurements obtained from manual dips when installing level-loggers following completion of investigations.

3.7 Geotechnical laboratory testing

The following laboratory testing was undertaken on samples collected from select HAs:

- 1 5 No. Atterberg Limits tests (including natural water content) – NZS4402 Tests 2.1 to 2.4.
- 2 5 No. Linear Shrinkage tests – NZS4402 Test 2.6.

A summary of the laboratory test results is provided in Table 3-7, with the corresponding laboratory test report presented in Appendix C.

Table 3-7: Summary of geotechnical laboratory testing results

Sample ID	Sample Depth (m bcgl)	Laboratory Material Description	Atterberg Limits			Water Content (%)	Linear Shrinkage (%)
			Plastic Limit, <i>PL</i>	Liquid Limit, <i>LL</i>	Plasticity Index, <i>PI</i>		
BH003	1.0 – 1.1	Silty CLAY, with some sand; greyish brown. Very soft, moist, high plasticity	24	56	32	22.2	13
BH011	0.5 – 0.8	Silty CLAY, with some sand; greyish brown, mottled orange. Firm, moist, high plasticity	40	127	87	44.7	22
BH018	0.3 – 0.8	Silty CLAY, with some sand; greyish brown. Stiff, moist, high plasticity	30	80	50	28.8	16
BH027	0.3 – 0.8	Silty CLAY, with some sand; dark brown, mottled orange. Firm, moist, high plasticity	28	70	42	32.0	16
BH034	0.5 – 1.0	SILT with some clay, some sand and trace of gravel; light grey, mottled brown. Soft, moist, low plasticity	29	56	27	35.3	12

3.8 General geological model

3.8.1 General

The ground conditions encountered within the site-specific investigations align with published geology (Section 3.1) and earlier ground models mentioned in the T+T Geotechnical Plan Change Report (Ref. [8]). The geological units encountered across the Site are summarised in Section 3.8.2 to Section 3.8.5 and presented in the appended Geological Cross-Sections in Appendix A. A tabulated summary of the geological model of the Site is presented in Table 3-8.

Table 3-8: General geological model

Geological Unit	General Soil Description	Depth to Top of Layer	Layer Thickness	Undrained Shear Strength, S_u	Cone Tip Resistance, q_c
Topsoil	Organic SILT, dark brown, stiff to very stiff, dry to moist, low plasticity	0 m	0.1–0.8 m (typically 0.2 m)	N/A	N/A
Historic Fill	Clayey SILT, mottled brown-orange and grey, very stiff to hard, moist, low to medium plasticity	0–2.5 m	0.2–5.2 m (unconfirmed in areas)	68–220+ kPa	0.2–4.0 MPa
Tauranga Group Soils	Sandy SILT some clay, dark orange/grey moist, stiff to very stiff, low to medium plasticity	0–0.8 m	0.8 m	81–115 kPa	N/A
Residual ECBF Soils	Clayey SILT / silty CLAY minor sand, orange and yellow with streaked grey, very stiff to hard, medium to high plasticity	0.2–5.2+ m	2.3–28.2 m	83–220+ kPa	0.3–8.0 MPa
ECBF Rock	Interbedded sandstone and mudstone, extremely weak	5.3–28.6 m	Unknown	N/A	10+ MPa

3.8.2 Topsoil

Topsoil depths across the Site ranged from 0.05 – 0.5 m thick, with a typical thickness of 0.2 m. It is typically described in the field investigations as stiff to very stiff.

3.8.3 Historical fill

Localised fill materials associated with the historical golf course earthworks were encountered in some investigation boreholes, extending to depths of up to 2 m below current ground level (bcgl). The majority of the fill encountered across the Site was generally less than 1 m thick and comprised reworked site-won ECBF soil, which were typically assessed as very stiff to hard. A thicker fill profile was encountered toward the eastern end of the Site (Employment development area), where the materials were more variable and included buried organics and greywacke gravels. Topsoil was encountered beneath the historic fill in some places, implying that historic golf course fill was placed directly atop the previous ground level.

3.8.4 Tauranga Group Soil / Colluvium

In general, Tauranga Group alluvial soils and colluvium within the Site are anticipated to be mainly confined to the gully areas and the low-lying coastal areas. Tauranga Group soils and colluvium that were encountered in the investigations consist of sandy silts to a depth of 2.0 m bcgl.

3.8.5 East Coast Bays Formation (Waitemata Group)

ECBF of the Waitemata Group underlies much of the Auckland isthmus. Soils weathered from the ECBF rock were widely encountered across the Site, below the surficial topsoil, Tauranga group/colluvium or historic fill. The ECBF soils generally comprised silts and clays, with occasional thin beds of sand or sandy silt, and were typically assessed as very stiff to hard.

ECBF weathered rock is generally expected to be present below 2.5 m to 20 m depth below natural ground level, comprising alternating beds of greenish grey muddy sandstone and siltstones. ECBF was proven to a depth of 179.5 m below ground level in a historical borehole by PDP in 2008.

3.8.6 Groundwater

Out of the 35 boreholes undertaken for the Project, groundwater was encountered in 29 of these boreholes during May 2026, with groundwater levels measured between 0.1 m to 5.0 m bcgl. The groundwater measurements carried out on site to date, combined with our general experience in this terrain and observations during the 2025-2026 Stage 1 earthworks in the north of the Site confirms that groundwater is perched within the interbedded clayey and sandy beds of the residual ECBF soils, typically building up 1 to 2 metres head as it cascades/seeps down to the deeper groundwater table which is controlled by the invert of surrounding gullies and the coastline. The perched groundwater lenses can be ephemeral (i.e. reducing in head during summer) and limited in volume and spatial extent.

As an example, perched groundwater that was encountered during Stage 1 earthworks in February 2026 near the golf course entrance encountered an initially strong flow of groundwater (Figure 3.3) when it was encountered mid-afternoon, but it reduced to a trickle by the following morning, i.e. confirming that the groundwater was perched and of limited volume. At this particular location, the ground level of approximately 27mRL is well above the elevation of the gully to the immediate north at 16mRL, dropping to the permanent stream at 8mRL.



Figure 3.3:

LEFT: Outflow of groundwater from an intercepted perched lens of groundwater in the Stage 1 area, February 2026. Flow was initially strong (as shown), but reduced to a trickle the next morning.

RIGHT: Photo location shown circled in red. Permanent streams shown in blue on the Council GIS.

Due to the variability of the perched groundwater regime at this Site, modelling of groundwater should be specific to the particular location and geohazard being assessed. Our groundwater modelling assumptions for assessment of liquefaction, slope stability and earthworks groundwater effects (and subsoil drainage) are presented in Section 4 and Section 5 of this report.

3.9 Geotechnical design parameters

A summary of the encountered subsurface materials and the adopted representative geotechnical properties is presented in Table 3-9.

The adopted properties for the various geological units underlying the Site are based on a combination of our previous experience working with similar materials under comparable ground conditions, a review of previous slope stability analysis from nearby earthworks sites in similar geological conditions (including the Stage 1 consented earthworks area), interpretation of the site-specific investigation data available across the Site (including both historic and recent investigations), and a review of the Site geomorphology.

Table 3-9: Summary of material properties

Geological Unit	Unit Weight (kN/m ³)	Effective Stress Shear Strength Parameters		Undrained Shear Strength	Elastic (Young's) Modulus
		ϕ' (degrees)	c' (kPa)		
Historic Fill	17	28	3	60	8
Engineered Fill	18	30	5	140	15
Tauranga Group Soil / Colluvium	17	26	3	40	5
Residual ECBF Soil	17	28	5	70	10
Highly Weathered ECBF Rock	19	32	10	200	40
Moderately – Slightly Weathered ECBF Rock	20	40	25	-	100

Note:

- (1) Where historic fill, Tauranga Group soil and colluvium are encountered, it should be inspected and verified by a suitably qualified Geotechnical Engineer during bulk earthworks to confirm that it is compatible with the adopted design parameters. Where it is assessed to be inadequate, it should be undercut and replaced with engineered fill.

4 Geohazards assessment

4.1 General

Section 106 of the Resource Management Act 1991 (Ref. [11]) requires an assessment of the risk from natural hazards to be carried out when considering the granting of a subdivision consent, and specifically states that the assessment must consider the combined effect of the natural hazard likelihood and material damage to land or structures (consequence). A hazard risk assessment is also required to be undertaken by AUP:OP Chapter E36.

The following sections of this report provide an assessment of the geohazards relevant to this Site, with reference to the AUP:OP (Ref. [3]) Chapter E36 (Natural Hazards and Flooding, as amended by PC120) and NPS-NH (Ref. [4]):

- 1 Site response to earthquake shaking and its effects (i.e. liquefaction risk and active faults);
- 2 Slope stability including the potential for landslides/landslips¹;
- 3 Expansive soils;
- 4 Existing non-engineered (historic) fill;
- 5 Settlement;
- 6 Erosion; and
- 7 Groundwater impact.

4.2 Seismic considerations

4.2.1 Recent updates to seismic hazard understanding

Recent developments in the understanding of seismic hazard in New Zealand include:

- 1 The release of the updated New Zealand National Seismic Hazard Model (NSHM, Ref. [12]) in October 2022.
- 2 The publication of TS 1170.5:2025 (Ref. [13]).

These updates reflect advances in scientific knowledge regarding earthquake sources and ground motion characteristics, which have implications for both structural and geotechnical design.

4.2.1.1 2022 National seismic hazard model

In October 2022, GNS Science released the updated NSHM (Ref. [12]), which represents the latest scientific understanding of earthquake hazard in New Zealand. While the NSHM (Ref. [12]) will inform future design standards and guidance documents, it does not provide information that can be directly applied in design. Consequently, the current minimum compliance pathway under the New Zealand Building Code (NZBC) remains unchanged.

4.2.1.2 TS 1170.5:2025

TS 1170.5:2025 (Ref. [13]) has been developed to incorporate the updated NSHM (Ref. [12]) and current understanding of earthquake risk into design practice.

While NZS 1170.5:2004 (Ref. [14]) remains the primary referenced standard for NZBC compliance, TS 1170.5:2025 (Ref. [13]) may be adopted as an alternative solution to demonstrate compliance using the latest seismic hazard parameters and engineering principles.

For this project, seismic hazard values have been adopted in accordance with MBIE Module 1 (Ref. [15]), which is consistent with the seismic hazard values and intent of TS 1170.5:2025 (Ref. [13]).

¹ The NPS-NH refers to “landslips”, and PC120 refers to “landslides”. These terms are considered to be interchangeable, they mean the same thing.

4.2.2 Seismic site subsoil class

Seismic accelerations to be resisted by a structure are influenced by the stiffness of the underlying soil and rock profile. Softer soil deposits can amplify ground shaking, resulting in higher seismic demands on structures founded on these materials.

Based on the results of the site-specific geotechnical investigations and our experience in the area, the Site is assessed to range from '**Site Class II**' to '**Site Class VII**' as per TS 1170.5:2025 (Ref. [13]), or '**Site Class C – Shallow Soil**' as per NZS 1170.5:2004 (Ref. [14]). The TS1170.2025 results incorporate a 30% range to the assessment results due to the lack of site-specific V_{s30} measurements. If required, and as part of detailed design, additional investigations could incorporate V_{s30} -specific investigations to reduce the 'Site Class' range.

In areas where significant excavation is proposed and ECBF rock is expected to be exposed at final levels, structures could be designed to '**Subsoil Class B – Rock**' as per NZS 1170.5:2004 (Ref. [14]), subject to building-specific geotechnical investigation and analysis.

4.2.3 Ground shaking hazard

The Site's seismic performance has been assessed with reference to the seismic shaking hazard and the requirements of the NZBC. Design earthquake actions are derived in accordance with NZS 1170.5:2004 (Ref. [14]) and are intended to meet the following design performance requirements:

- 1 Serviceability limit state (SLS) – The structure should suffer little or no structural damage. There may be minor damage to the structure that is readily repairable.
- 2 Ultimate limit state (ULS) – The structure is expected to suffer moderate to significant structural damage, but is not expected to collapse.

Table A1 of MBIE Module 1 (Ref. [15]) defines ground motions for soil earthquake response across New Zealand in terms of earthquake moment magnitude (M_w) and peak horizontal ground acceleration (PGA_H) for specified return periods derived from Table 3.3 of NZS1170.0:2002 (Ref. [16]). The design earthquake ground motions adopted for analysis are presented in Table 4-1, assuming Importance Level 2 (IL2) structures with a minimum design life of 50 years in accordance with NZS1170.0:2002 (Ref. [16]) and the Building Act 2004. Alternatively, if the TS 1170.5:2025 (Ref. [13]) ground shaking hazard values are to be adopted, further site investigations incorporating V_{s30} measurements of the soil and rock should be undertaken.

Table 4-1: Ground shaking hazard for geotechnical assessment

Seismic Design Model	Return Period (Years)	Peak Horizontal Ground Acceleration (PGA_H)	Earthquake Moment Magnitude, M_w
MBIE Module 1 (Ref. [15]),	500 (ULS)	0.19g	6.5
Site Class C – Shallow Soil	25 (SLS)	0.05g	5.9

4.2.4 Liquefaction

Soil liquefaction is a process that occurs when typically loose, saturated, low-plasticity soils (such as sands and silty sands) are subjected to strong cyclic loading during an earthquake. Some plastic soils may also be susceptible (typically low PI and high moisture content, Ref. [17][18]). Seismic shaking causes generation of excess pore water pressures that reduce the soil effective stress, leading to a loss of shear strength and stiffness. Liquefaction can result in bearing capacity failure, excessive settlement, and/or horizontal movement of soil masses (lateral spreading).

The vulnerability of a site to liquefaction depends on factors including soil density and plasticity, age of the soils, groundwater level, and the intensity and duration of earthquake shaking. The level of earthquake shaking at which liquefaction occurs is commonly referred to as the “trigger” PGA_H value. Denser soils generally require more intense and/or a longer duration of shaking than looser soil to liquefy (i.e. higher trigger thresholds).

T+T has evaluated the liquefaction susceptibility of the Site using the trigger methods recommended by Boulanger & Idriss (Ref. [19]). A liquefaction analysis has been undertaken using all 23 CPTs for the SLS and ULS design cases presented in Table 4-1 and assuming a groundwater level of 2 m bcgl. Analysis results are presented in Appendix D, and summarised in Table 4-2.

The liquefaction analysis indicates the following:

- 1 Liquefaction is not expected to occur during an SLS seismic event.
- 2 Liquefaction is triggered under ULS conditions.

Calculated consequence from liquefaction across the Site due to a ULS event is generally assessed as ‘insignificant’ (‘L0’) to ‘mild’ (‘L1’), resulting in negligible deformation of the ground and small settlements.

Calculations indicate that CPT-003, CPT-006 and CPT-019 result in ‘moderate’ (‘L2’) consequence under ULS shaking aligning with relatively small differential settlements. The calculation adopts a conservative crust thickness based on thin bands of material liquefying near to the ground surface. It is likely a thicker crust thickness will be present under ULS conditions. Corresponding consequence of liquefaction is expected to be more in the range of ‘insignificant’ (‘L0’) to mild (‘L1’).

Calculations indicate that CPT-014 aligns with a ‘high’ (‘L3’) consequence aligning with transient lateral displacements, moderate-to-large differential movements, and settlement of the ground in the order of 100 mm to 200 mm. This calculation is expected to be conservative and actual effects are likely to be less than this, due to the cementation and age of these materials, residual fabric and plasticity of the soils. This location is proposed for terrace housing (“CM4”) and a hotel (“CM5” as shown on the architectural drawings), and building-specific geotechnical investigations should be carried out as part of the typical design development process required for building consent:

- 1 As part of geotechnical investigations for building consent design, a cored machine investigation borehole should be considered, to characterise the underlying soils at this location.
- 2 Nested piezometers should be considered, to measure water pressures at depths which are theoretically susceptible to liquefaction (e.g. 9 to 10 m bcgl). The analysis described above assumed a hydrostatic groundwater profile, which is unlikely to be the case at this Site. The proposed pond at this location should also be taken into account in the assessment.
- 3 Soil samples should be taken from potentially liquefiable layers (e.g. 9 to 10 m bcgl) for Atterberg Limits laboratory testing, to check whether the soil is sufficiently non-plastic to liquefy.

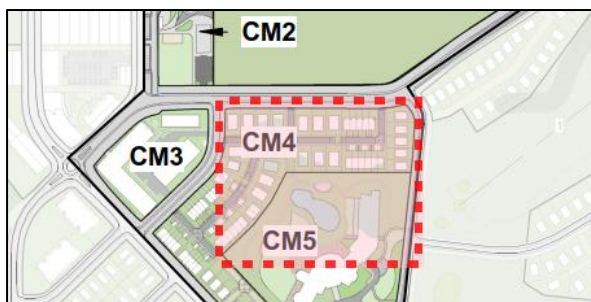


Figure 4.1: Supplementary site investigations are recommended in the CM4/CM5 area to confirm the liquefaction risk.

Table 4-2: Summary of CPT based liquefaction analysis for the ULS design case

CPT ID	Settlement – SV1D (mm)	Total liquefiable thickness – CTL (m)	Crust thickness from ground level (m)	Liquefaction Potential Index (LPI)	Liquefaction Severity Number (LSN)
CPT-001	1	0	14.8	0	0
CPT-002	49	2.7	3.8	2	6
CPT-003	65	2.8	3.5	3	10
CPT-004	49	2.5	3.0	2	5
CPT-005	26	0.7	3.5	0	6
CPT-006	73	2.7	3.9	3	10
CPT-007	42	1.9	3.4	1	4
CPT-008	58	2.7	4.7	2	8
CPT-009	70	2.9	5.2	3	7
CPT-010	16	0.4	9.8	0	3
CPT-011	66	2.4	3.4	4	8
CPT-012	25	0.7	4.6	0	3
CPT-013	6	0.2	7.2	0	1
CPT-014	181*	7.3	3.4	9*	19*
CPT-015	16	0.3	13.4	0	3
CPT-016	30	1.8	2.8	1	6
CPT-017	51	2.2	4.1	3	8
CPT-018	25	0.8	4.6	0	4
CPT-019	71	3.3	3.3	4	10
CPT-020	2	0	5.7	0	1
CPT-021	20	0.8	3.2	1	5
CPT-022	9	0.6	4.2	0	2
CPT-023	6	0.2	12.6	0	1

* This is likely to be conservative. Geotechnical investigations and tests should be carried out at building consent stage to confirm liquefaction risk at this site (proposed for terrace housing), as discussed in Section 4.2.4 of this report.

Note: LSN and LPI values typically represent the following consequences (From MBIE Module 3 Table 5.1 (Ref. [20]):

1. L0, Insignificant (LSN <10, LPI=0): No significant excess pore water pressures (no liquefaction).
2. L1, Mild (LSN 5-15, LPI=0): Limited excess pore water pressures; negligible deformation of the ground and small settlements.
3. L2, Moderate (LSN 10-25, LPI <5): Liquefaction occurs in layers of limited thickness (small proportion of the deposit, say 10 percent or less) and lateral extent; ground deformation results in relatively small differential settlements.
4. L3, High (LSN 15-35, LPI 5-15): Liquefaction occurs in significant portion of the deposit (say 30 percent to 50 percent) resulting in transient lateral displacements, moderate-to-large differential movements, and settlement of the ground in the order of 100 mm to 200 mm.

4.3 Slope stability and landslide risk

4.3.1 General

The ECBF soil and rock underlying the Site is known to comprise variable, weakly cemented sandstones, siltstones, and mudstones, which are susceptible to weathering and erosion, particularly when exposed and saturated. These conditions can reduce material strength and increase the risk of slope failure under rainfall or seismic loading.

4.3.2 Coastal areas

Areas of the Site are located near the coastline. An assessment of the potential impact of coastal instability and erosion on the proposed development has been undertaken and is presented in the T+T Coastal Hazard NPS-NH and AUP Risk Assessment Report (Ref. [21]).

4.3.3 Historic instability

Historic slope instability is evident in isolated parts of the Site, as described below.

4.3.3.1 Southern coastal slopes

A slip feature is located at the south-western edge of the proposed Coastal sub-precinct (Figure 4.2), where a semi-circular evacuated feature is evident. The geomorphology suggests this may be a drainage pathway, which on regression has instigated mass movement. Stability of the slopes along the coastline are discussed in Section 4.3.7 of this report. This slope has been modelled as Section 6.



Figure 4.2: Historical landslide at the south-western edge of the proposed Coastal sub-precinct

4.3.3.2 Near Jack Lachlan Drive (outside the Site)

A cut slope outside the Site near Jack Lachlan Drive (Figure 4.3) is proposed to remain in use as a golf course fairway. It is likely that this instability resulted from golf course development and the surcharge from the adjacent road. This instability is not assessed to affect the Project.

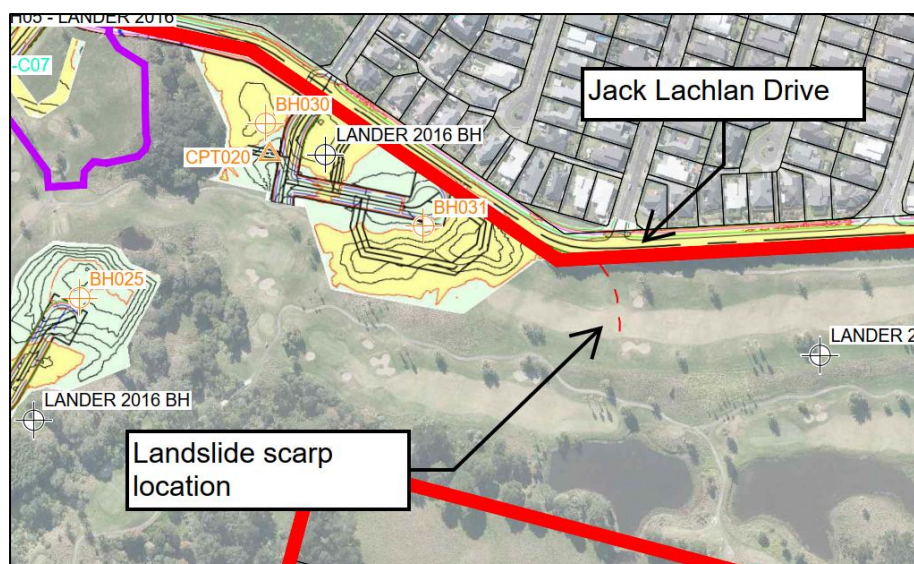


Figure 4.3: Location of a landslide headscarp observed near Jack Lachlan Drive

4.3.4 Slope stability criteria

The stability of slopes under various design conditions is assessed using a Factor of Safety (“FoS”), defined as the ratio of resisting forces to driving forces. The following performance criteria for slope stability assessment are recommended in accordance with the Auckland Code of Practice for Land Development and Subdivision (Ref. [5]):

Table 4-3: Slope stability factor of safety

Design Condition	Factor of Safety against slope instability		
	Building platform, roads, buried services	Services & maintenance access	Amenity, parks, bush
Static long-term conditions (drained soil conditions, normal groundwater)	1.5	1.4	1.2
Transient short-term conditions (drained soil conditions, worst credible groundwater)	1.3	1.2	1.1
ULS seismic condition (undrained soil conditions, normal groundwater, with a PGA_H of $0.19g^{(2)}$)	1.0	1.0	1.0

Notes:

- (1) Definitions of terms as provided in the Auckland Code of Practice (Ref. [5]).
- (2) The adopted PGA_H value has been derived in accordance with MBIE Module 1 (Ref. [15]).

4.3.5 Soil parameters

The soils are predominantly cohesive silts and clays, which are expected to behave in an undrained manner during short-term seismic loading and for a short period following initial excavation. Undrained soil shear strengths (S_u), used to assess slope stability under seismic conditions, were derived from handheld shear vane test results.

The adopted design soil parameters are summarised in Table 3-9 (Section 3.9).

4.3.6 Imposed loads

A nominal uniformly distributed surcharge of 12 kPa and 15 kPa have been adopted in the slope stability models to represent general imposed loading associated with buildings and traffic, respectively. Any significant changes to the proposed development layout, loading conditions, or ground levels may require reassessment of the slope stability analyses.

4.3.7 Slope stability analyses

Slope stability analyses were undertaken using the software Slope/W, employing the Morgenstern-Price method with a half-sine inter-slice force function to satisfy both moment and force equilibrium. Critical failure surfaces were identified using the grid-&-radius method and/or entry-exit method to evaluate a comprehensive range of potential failure surfaces. In all cases, the most critical failure surface was optimised to determine the minimum FoS.

Seismic loading for slope stability assessment was assessed in accordance with NZS 1170.5:2004 (Ref. [14]) and the NZTA Bridge Manual SP/M/022 (Ref. [22]). A PGA of 0.19g was adopted for the stability analyses.

Based on site investigation data and our experience in this terrain, stability analyses were undertaken for two groundwater scenarios:

- 1 *A worst credible groundwater table* assumed at between 1-4 m depth below the surface of the residual soil profile, and
- 2 *A normal groundwater condition*, rising from the base of nearby gullies. As shown in the slope models in Appendix A, hydrostatic groundwater has typically been modelled at 2 m to 7 m bgl across the majority of the Site. This is considered to be a conservative representation of the absence of groundwater in the majority of the investigation boreholes, and the occasional presence of perched groundwater.

The results demonstrate that the proposed earthworks design contours generally achieve slope stability FoS that meet or exceed the criteria set out in Table 4-3. During construction, any unsuitable materials encountered at finished subgrade level (e.g. soft or excessively wet soils) should be undercut and replaced with suitable engineered fill to ensure the design assumptions and long-term slope performance remain valid.

Where retaining walls are proposed, detailed wall design and site-specific stability analyses will be required at a later stage, to demonstrate compliance with the Auckland Code of Practice (Ref [5]). Depending on the confirmed ground conditions, stabilisation measures such as wall embedment or undercutting prior to wall construction may be required to achieve acceptable slope stability performance.

For some parts of the southern cliff area (as identified on Geological Cross-Sections 7, 8 and 9), a reinforced concrete palisade pile wall will be required to provide long-term stabilisation of the slope and achieve the target FoS. The indicative location and extent of the wall, including a typical detail, are shown in Figure 1014358.3200–Figure 12 in Appendix E.

A summary of the slope stability results is presented in Table 4-4 below. Detailed stability outputs are attached in Appendix E.

Commentary on temporary slopes is provided in Section 5.2.6 of this report.

Table 4-4: Slope stability analyses results

Analysed Location	Slope Stability Factor of Safety			Comments
	Static (Target FoS = 1.5)	Transient (Target FoS = 1.3)	Seismic (Target FoS = 1.0)	
XS-1	>1.5	>1.3	1.7	The Site is approximately 18 m from the crest of the cliff (and the nearest lot is approximately 30 m from the crest of the cliff).
XS-2	1.6	1.4	1.5	Slope is fully within the reserve.
XS-3	2.0	2.0	1.6	Fill is proposed on an existing slope, which is inferred to contain colluvium. Inspection of the stripped surface by a Geotechnical Engineer will be required prior to filling, to confirm ground conditions match the assumed model ⁽²⁾ .
XS-6	1.7	1.5	1.3	This analysis is through the relic landslide feature. Preliminary analysis indicates the slope is now stable, but this should be confirmed by site-specific geotechnical investigation and analysis [recommended condition of consent⁽¹⁾].
XS-7	1.5	1.3	1.3	Preliminary design indicates an 18 m deep pile wall at the edge of the earthworks fill will be necessary to achieve acceptable FoS (subject to design refinement [recommended condition of consent⁽¹⁾]).
XS-8	1.5	1.4	1.1	Preliminary design indicates an 8 m deep pile wall at the edge of the earthworks fill will be necessary to achieve acceptable FoS (subject to design refinement [recommended condition of consent⁽¹⁾]).
XS-9	1.5	1.3	1.0	Preliminary design indicates a 14 m deep palisade pile wall at the edge of the earthworks fill will be necessary to achieve acceptable FoS (subject to design refinement [recommended condition of consent⁽¹⁾]).

Analysed Location	Slope Stability Factor of Safety			Comments
	Static (Target FoS = 1.5)	Transient (Target FoS = 1.3)	Seismic (Target FoS = 1.0)	
XS-11	1.8	1.5	1.6	Fill is proposed on an existing slope. Inspection of the stripped surface by a Geotechnical Engineer will be required prior to filling, to confirm ground conditions match the assumed model ⁽²⁾ .
XS-12	1.9	1.8	1.8	Existing/historical fill is expected in this area. Inspection of the stripped surface by a Geotechnical Engineer will be required prior to filling, to confirm ground conditions match the assumed model ⁽²⁾ .
XS-15	2.1	1.9	1.6	Analysis at the centre of the gully to be filled for development of the Employment sub-precinct. The gully will need to be mucked out as agreed on site with the Geotechnical Engineer, prior to placement of any subsoil drainage and fill ⁽²⁾ .
XS-18	>1.5	>1.3	1.5	The Site is approximately 18 m from the crest of the cliff (and the nearest lot is approximately 38 m from the crest of the cliff).
XS-19	>1.5	>1.3	>2.1	The nearest lot is approximately 21 m from the crest of the cliff.
XS-20	>1.6	>1.6	2.2	The nearest lot is approximately 30 m from the crest of the cliff.

Notes:

- (1) Recommended conditions of consent are listed in Section 7.1 of this report.
- (2) This is a typical requirement for earthworks (along with the recommendations presented in Section 5.2), and is not required as a specific consent condition.

4.4 Expansive soils

Expansive soils (or “reactive soils” using Australian terminology) are clay-rich soils that undergo appreciable volume changes in response to moisture fluctuations. These soils swell when wet and shrink when dry, leading to vertical ground movements that can damage structures. This hazard is best managed through a combination of appropriate site preparation and careful foundation design. Guidance is provided in Section 5.3.2 of this report.

4.5 Existing non-engineered (historic) fill

Most of the localised historic fill associated with golf course earthworks was found to comprise very stiff to hard reworked ECBF, and is therefore not expected to present a significant risk of consolidation settlement or instability associated with the proposed development and earthworks. However, given that certification records of the historic golf course earthworks at the Site are not available, the condition of any areas of existing fill exposed will need to be inspected by a suitably qualified

Geotechnical Engineer during earthworks to verify that the exposed conditions are compatible with the assumed model presented in this report (as noted in Section 5.2.4 of this report). If weak, compressible or unstable soils incompatible with the assumptions presented in this report are found, they will need to be undercut and replaced with engineered fill. Depending on the composition of the existing fill, it may be possible to re-use it as engineered fill. We recommend these requirements be included as consent conditions.

The site investigations indicate that deeper golf course existing fill is present in the Community development area near CPT-014, surrounding the existing clubhouse / residential dwellings, and throughout the front nine of the golf course (Employment and Central East Precinct). In these areas, historical cut-and-fill operations appear to have been undertaken within several gulleys to create fairways, greens, lakes, and associated landscaping features.

The existing fill encountered in the area of CPT-014 comprises stiff to very stiff soils extending to approximately 1.5 m depth. Proposed earthworks in this area include up to 3 m of filling and 2 m of cutting. It is anticipated that existing fill in this part of the Community area can be left in place, provided that inspections during earthworks verify that the existing fill encountered is compatible with the conditions described in this report.

Surrounding the clubhouse and residential dwellings, historic fill comprising very stiff to hard soils was encountered to depths of up to approximately 2.15 m. The fill commonly contains gravel- to cobble-sized material and locally includes significant organic matter. Buried topsoil was also encountered in several locations. The Project proposes to redevelop the existing clubhouse and construct a seven-storey hotel within this area. There is a likelihood that the existing fill within this area will require partial or complete removal to enable development of these structures.

The existing fill encountered in the Employment area of the Site comprises typically stiff soils extending to 1.5 m deep. It is expected that the existing fill profile will be thicker closer to the road. However, the existing fill material is more variable with inclusions of gravel and organic topsoil within the existing fill as noted in the investigations. As this area is in the location of significant cut and fill (locally up to 9 m of fill and 5 m of cut), inspections during the cut process should be undertaken to confirm the strength and material profile of the existing fill in the area. We recommend these requirements be included as consent conditions (as noted in section 7.1 of this report). There is the potential that the existing fill may have to be removed from below the proposed fill areas.

Within the Central East Precinct, the existing fill comprises of typically very stiff soils extending to approximately 2.2 m depth and is commonly underlain by buried topsoil. Some investigation locations encountered fill containing significant proportions of organic-rich material. It is inferred that this existing fill is associated with earthworks to develop the eighth and ninth holes. As up to 2 m of additional filling is proposed in parts of this area, removal and replacement of existing fill may be required where unsuitable materials are encountered beneath proposed fill platforms.

4.6 Settlement

The placement of earth fill may induce some degree of consolidation within the underlying soils. In areas where soft or wet soils are encountered, these materials must be undercut and replaced with suitable engineered fill to minimise post-construction settlement.

In general, settlements of less than 10 mm are expected beneath the weight of earthworks fills. The settlement analysis outputs are provided in Appendix F. The majority of settlement is expected to occur relatively rapidly during placement of earthworks. If deeper fill is proposed, or if thicker or softer compressible soils are encountered, settlement monitoring points on the finished surface should be considered, for monitoring following construction to verify that settlement rates remain within acceptable limits for proposed residential development. The number, location, and monitoring frequency of settlement monitoring points will be determined in consultation with the Geotechnical

Engineer during construction. Additional requirements and monitoring procedures are outlined in the Draft Geotechnical Works Specification presented in Appendix G.

4.7 Erosion

Where surface water flows over steep slopes, it can lead to erosion of topsoil and underlying soils. As is standard practice in earthworks design, civil works and building platforms should be designed to direct surface water (runoff) away from batter slopes and into the reticulated stormwater system or designated outlet structures within vegetated or bush areas, to mitigate the risk of erosion.

4.8 Groundwater impact assessment (earthworks)

Most of the earthworks cuts are less than 5 m deep, and well within the Site boundaries. The deepest excavation is 9 metres deep for a pond at the north of the site, adjacent to the existing golf course entrance (Figure 4.4, below). The base of the proposed pond is approximately 34 metres from the Site boundary at Jack Lachlan Drive. The base of the pond will be at 14.5mRL, discharging to the stream gully to the east at approximately 13.0mRL.

As discussed in section 3.8.6 of this report, it is expected that lenses of perched groundwater may be encountered in the sides of the pond excavation. Seepages from perched groundwater lenses are expected to be similar to those observed during the Stage 1 earthworks which were carried out during 2025 – 2026 to the west of the golf course entrance road (at the left of Figure 4.4); in which outflows of perched water are limited in volume and pressure head, and tend to reduce to a trickle within a matter of days.

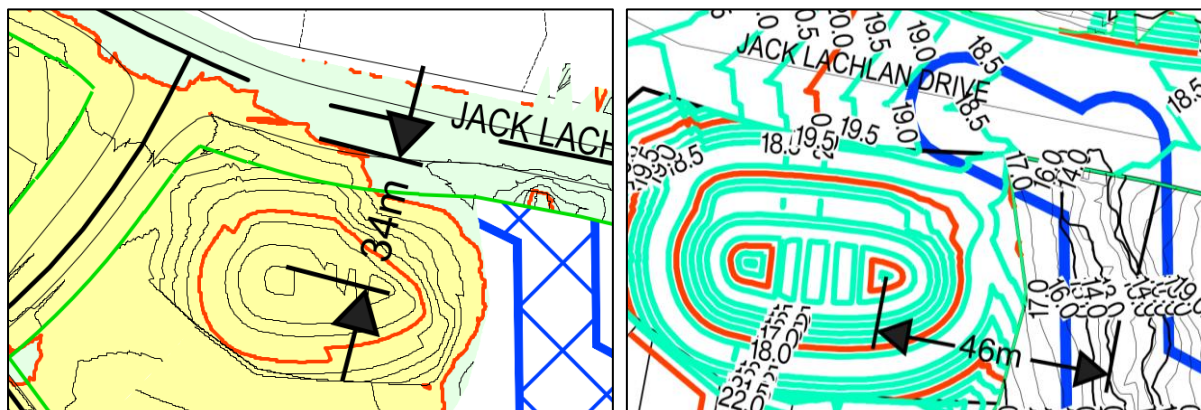


Figure 4.4: Proposed pond adjacent to the existing golf course entrance.

LEFT: Cut/fill 0.5m contours (showing approximately 9 m depth of cut to form the pond)

RIGHT: Proposed ground surface 0.5m contours (green & red) and existing 1m contours (black)

As the pond discharges to the stream to the east, it will not drain groundwater from that gully. Nonetheless, given that this excavation is the deepest and the closest to a Site boundary, we have carried out a calculation of groundwater drawdown based on the following conservative assumptions:

- Groundwater drawn down to the base of the pond (though it is not intended to dry out);
- Existing hydrostatic groundwater near ground surface level at the boundary (though groundwater is actually perched, overlying a wider groundwater table controlled by the stream gully at approximately 10 m depth);
- Distance from the pond base to the Site boundary of approximately 32 metres;
- Conservative horizontal permeability of $k_h = 1.5 \times 10^{-7}$ m/s in the residual ECBF soil.

A simple steady state inflow calculation (Preene, Roberts, Powrie 2016) indicates this would cause a drawdown radius of approximately 22 metres, i.e. groundwater drawdown effects will be wholly within the Site boundaries, even based on this very conservative scenario.

As this is the deepest proposed excavation closest to a Site boundary, it can therefore be concluded that *the natural groundwater level will not be reduced by more than 2m on the boundary of any adjoining site*; i.e. the proposed earthworks meet Permitted Activity criteria in section E7.6.1.10(3) of AUP:OP (Ref. [3]).

Building with basements are also proposed for the Site – this is discussed in Section 5.5 of this report.

4.9 Natural hazard assessment

4.9.1 General

The preceding sections of this report assess the geotechnical hazards relevant to the Site, including liquefaction, slope instability and coastal hazards.

Coastal erosion, coastal inundation and tsunami hazards are addressed separately in the T+T Coastal Hazard NPS-NH and AUP Risk Assessment Report (Ref. [23]), while flooding hazards are addressed in the relevant hydrological assessment report prepared by Harrison Grierson Consultants Limited (HG, Project Civil Designer).

The following sections provide a summary assessment of natural hazard risk in the context of the AUP:OP (Ref. [3]) Chapter E36 (Natural Hazards and Flooding, as amended by PC120) and NPS-NH (Ref. [4]), which adopt a risk-based approach to subdivision and development. The assessment considers the nature of the identified hazards, their likelihood and consequences, and the extent to which the proposed development avoids, mitigates, or manages the associated risks.

4.9.2 Landslide hazard

The Project development is generally well set back from the coastal cliffs and steeper gullies. Slope stability analyses demonstrate that the proposed earthworks can achieve the Auckland Code of Practice [Ref. [5]] required FoS under static, transient and seismic loading conditions. Where required, mitigation measures including setbacks, earthworks controls, retaining structures, engineered fill placement and geotechnical verification during construction can be implemented to manage localised instability risks.

In terms of the *likelihood* of landslides: the Council Code of Practice factor of safety requirements do not directly reference annual exceedance probabilities (as required by NPS-NH, Ref. [4]); however according to the correlation published in the New Zealand Geotechnical Society Slope Stability Guidance documents (Ref. [24]) (Figure 10.3, Silva et al 2008), where slopes have been assessed to achieve factors of safety of 1.5 complying with Council requirements, the likelihood of failure is **rare** (approximately 0.01% AEP).

In terms of *consequence*: land development is proposed on ground that is generally flat or gently sloping; and accordingly any slope instability in this ECBF terrain would be expected to occur gradually and slowly, i.e. ground movements would typically provide forewarning of potential issues, with the possibility of remediation or prevention of damage to buildings or infrastructure. No development is proposed near the steeper sea cliffs at the west of the Site, where landslides could develop more rapidly. Furthermore, no development is proposed near the base of any steep slopes, i.e. within any potential landslide run out zones. Accordingly, the consequences of landslides at this Site are most likely to fall with the Negligible to Moderate levels (as defined by NPS-NH, Ref. [4]), resulting in an overall **low landslide hazard**.

In terms of PC120, both the large-scale and shallow landslide susceptibility maps (Figure 4.5) indicate that the Site generally presents a low to moderate landslide hazard prior to earthworking (with the exception of the gully area at the south, which will be set aside as an EPAN and will not be developed). The areas where moderate to very high shallow instability are indicated have been specifically analysed in Section 4.3 of this report. The analyses have identified that:

- 1 The crests of the western cliffs are by their nature inherently marginally stable (as signalled by the PC120 shallow instability map); but the development is set well back from these cliff edges (refer to XS-1, XS-18, XS-19, XS-20 in Table 4-4), such that the resulting landslide risk to the development is low and meets NPS-NH (Ref. [4]), PC120 and the Auckland Code of Practice (Ref. [5]) criteria.
- 2 Earthworks will be carried out to modify the landform across much of the Site, and the resulting slopes will meet Council Code of Practice requirements, with resulting low risk of landslides (detailed in Section 4.3 of this report). Engineering measures are proposed for the southern cliffs (refer to XS-7, XS-8, XS-9 in Table 4-4), and a resource consent condition has been recommended to confirm stabilisation requirements at this location (listed in Section 7.1 of this report).
- 3 Remaining areas with moderate to very high risk of shallow landslides (as indicated by the PC120 shallow instability map) are confined to the gullies which are set aside as EPAN.



Figure 4.5: Auckland Council PC120 landslide hazard maps (Left: large-scale; Right: shallow instability)

4.9.3 Liquefaction

Liquefaction assessment indicates that liquefaction is not expected under SLS shaking and may occur locally under ULS shaking conditions. Across the majority of the Site, predicted liquefaction consequences range from insignificant (L0) to mild (L1), with isolated areas indicating moderate (L2) and locally high (L3) calculated consequences.

The higher consequence results are considered conservative and are associated with limited areas of localised alluvial soils. Furthermore, proposed earthworks are expected to remove, regrade, or improve some of these deposits through placement of engineered fill.

Overall, liquefaction is not expected to present a significant constraint to the proposed development. **The residual liquefaction risk is considered low** and can be appropriately managed through detailed design and standard geotechnical construction practices. As noted in section 7.4 of this report, additional investigations are recommended in the CM4 and CM5 areas (near CPT-014), to confirm the liquefaction hazard at that location.

4.9.4 Active faults

No active fault traces are mapped within the Site, and no active fault avoidance overlays apply under the AUP:OP (Ref. [3]). Although regional geological evidence indicates historical tectonic deformation within the wider area, no evidence of active surface fault rupture affecting the Site has been identified during the investigations or from available geological mapping.

Given the absence of a mapped active fault crossing the Site, **the risk of surface fault rupture affecting the proposed development is considered low.**

Active faulting is therefore not considered a constraint to the proposed subdivision development.

5 Geotechnical considerations

5.1 Overview

This section discusses the key geotechnical considerations relevant to the proposed Site development and presents the supporting recommendations. The matters addressed include:

- 1 Earthworks (including temporary stability);
- 2 Foundation considerations;
- 3 Retaining walls; and
- 4 Excavations for buildings and basements.

The recommendations and opinions in this report are based on the results of discrete site investigations undertaken. The nature and continuity of the subsurface conditions away from these investigations is inferred and actual conditions may vary from the assumed model.

5.2 Earthworks

5.2.1 General

All earthworks must be carried out in general accordance with the requirements of the Auckland Code of Practice for Land Development and Subdivision (Ref. [5]) and NZS 4431:2022 (Ref. [25]), under the supervision of a Chartered Professional Geotechnical Engineer familiar with the contents of this report.

Erosion and sediment control measures should be in accordance with the Auckland Council General Guidance Document (GD05) for Sediment and Erosion Control (Ref. [26]).

A Draft Geotechnical Works Specification is provided in Appendix G, which outlines requirements for site preparation, fill placement, subsoil drainage, compaction requirements, quality assurance testing, and as-built requirements.

Those requirements are further summarised in the following sub-sections.

5.2.2 Earthworks operations

Cuts of up to approximately 5 m and fills of up to approximately 9 m (per HG cut/fill plan presented in Appendix H) are required to achieve design levels (localised deeper cuts are proposed for ponds in the north of the Site). The predominant cut material is ECBF soils, which is generally suitable for reuse as engineered fill.

Any unsuitable materials (e.g. soft or excessively wet soils) encountered at design subgrade must be undercut and replaced with certified engineered fill. All undercut areas and pre-existing fill stripping must be inspected by a suitably qualified Geotechnical Engineer to ensure all unsuitable materials are removed.

Earthworks should be carefully planned to manage varying soil types, allowing for soil conditioning where necessary to meet engineered fill specifications.

Materials with high organic content (e.g. topsoil, isolated alluvium) are unsuitable for engineered fill and must be removed off-site or placed in landscaping / reserve areas with no anticipated future development. Inorganic waste such as rubbish and demolition debris must be removed off-site.

Vegetation within earthworks areas must be cleared, with minimal disturbance beyond earthworks extents to control silt. Topsoil should be stripped from all cut and fill areas, extending beyond

earthwork boundaries to avoid contamination. Tree roots must be removed to approximately 0.5 m below ground where trees are cleared.

5.2.3 Excavatability

A large proportion of the proposed cuts are expected to be within residual ECBF soils, although some areas may encounter weathered ECBF rock within the upper 10 m of the profile. The weathered ECBF is typically extremely to highly weathered and comprises extremely weak to very weak rock material. Based on the investigation results and our experience with similar materials in the area, both the residual soils and highly weathered rock can generally be excavated using standard earthworks plant such as scrapers, bulldozers, and excavators. Excavation within these soils is not expected to require specialist plant under typical conditions.

In deeper cuts, there is potential to encounter more competent moderately to slightly weathered ECBF rock. While often weathered and fractured enough to allow excavation with conventional equipment, some zones of less weathered or cemented rock may require the use of specialist ripping equipment, such as a 20-tonne excavator with a toothed bucket or a bulldozer fitted with a rock ripper.

5.2.4 Verification of ground conditions and subgrade preparation

Exposed soils should be inspected by a suitably qualified Geotechnical Engineer after vegetation and topsoil is stripped, to confirm that ground conditions are compatible with the assumed model. Vegetation, topsoil, soft ground and any unsuitable material must be stripped prior to placing fill.

Benching of sloped ground is required prior to placement of fill on slopes.

The Contractor is responsible for planning and sequencing construction activities to ensure the integrity and condition of the prepared subgrade are maintained at all times.

5.2.5 Groundwater and subsoil drains

Perched groundwater is expected to be present below some parts of the Site.

As shown on the HG Earthworks drawings (Appendix H), proposed earthworks bulk cuts are generally up to 5 m deep, and generally 1 m to 3 m (in areas of cut). Localised seepages may be encountered in some cuts, and they may need to be intercepted with subsoil drains to allow development to proceed. Subsoil drainage will be also required at the invert level of any gullies or wet ground encountered during earthworks. Installation should follow the stripping of topsoil and removal of soft materials. Interception of perched groundwater at the base of existing/historical gullies will be particularly beneficial in areas such as the Village Centre, where basements are proposed to be excavated.

The subsoil drains consist of 160 mm diameter Hi-Way grade perforated pipes, installed approximately 1.0 m below the invert level and surrounded by SAP50 drainage scoria (or an approved equivalent drainage aggregate product). A typical subsoil drainage detail is included in Appendix I.

All groundwater collected by the subsoil drainage system must be discharged into an appropriate swale or stormwater drainage network. Where necessary, formal outlet structures designed to prevent scour erosion should be constructed at the drain outlets. Following installation, the drainage system should be flushed to verify performance and then maintained regularly through a controlled monitoring program.

Throughout construction, the position and elevation of all subsoil drains and flushing points should be surveyed and recorded on as-built drawings. Adequate provision for subsoil drainage must be made where elevated groundwater levels could adversely impact civil works, such as road subgrades and future building platforms.

5.2.6 Temporary slopes

Temporary cut batters will be required throughout the Site to facilitate bulk earthworks and where necessary, retaining wall construction. These excavations will predominantly encounter residual ECBF soils, which are typically firm to very stiff but can vary in consistency and strength across the Site.

Temporary cut batters should generally be formed at a maximum gradient of 1V:2H unless subsequent works are scheduled to follow immediately after excavation. In low-risk or shallow excavation areas, steeper temporary batter angles may be considered, subject to site-specific geotechnical assessment. As a preliminary guideline, temporary slopes of up to 60° may be feasible for short-term excavations (less than one month) up to 3 m in height in dry, stable ECBF soils.

Where batters are proposed in existing fill, alluvial soils, or other non-cohesive materials, the risk of instability is expected to be greater. In these areas, temporary cut batters should generally be formed at a maximum gradient of 1V:3H unless otherwise approved by the Geotechnical Engineer. T+T should be consulted during construction to assess the stability of temporary batters formed in these materials and to provide project-specific recommendations where steeper slopes are proposed.

To ensure safe and stable temporary slopes, the following measures must be implemented:

- 1 Temporary batters should not remain exposed longer than necessary and should be stabilised, protected, or backfilled as soon as practicable. Where temporary batters are expected to remain open for extended periods, their stability should be reviewed by the Geotechnical Engineer.
- 2 Construction traffic and other surcharges (e.g. stockpiles) must be kept at least 1.5 m back from the crest of all temporary slopes.
- 3 No fill placement or stockpiling should occur directly above excavation areas.
- 4 All exposed batters must be protected from erosion and inclement weather where necessary using polyethylene sheets (or an approved equivalent product).
- 5 Surface water must be diverted away from batter crests to prevent erosion, softening and potential instability of exposed faces during rainfall events.
- 6 Excavation, drainage installation, and backfilling works must be staged carefully and closely coordinated.
- 7 Access to, and activities at, the base of temporary batters should be minimised. Where work is not actively being undertaken, these areas should be clearly delineated (e.g. temporary fencing or exclusion zones) to prevent unnecessary personnel access.

The Contractor is responsible for planning and sequencing construction activities to maintain the safety and integrity of temporary cut slopes. All proposed construction methodologies, including temporary batter angles and erosion protection measures, must be reviewed and confirmed with the Geotechnical Engineer prior to excavation. Ongoing monitoring of batter conditions during construction is essential, and any signs of instability or unexpected ground behaviour must be reported and reassessed immediately.

5.2.7 Excavation safety

A key design safety concern during excavation works is the potential collapse of temporary batters. To minimise this risk, no surcharging, fill or stockpiles should be placed above excavation areas before and during works. Excavations should ideally be carried out during dry weather conditions, with staged excavation recommended to maintain overall site stability.

The condition and stability of excavations must be continuously monitored by a suitably qualified Geotechnical Engineer throughout construction.

Additionally, careful consideration should be given to safely constructing drainage and other necessary tasks within the excavation, ensuring the safety of personnel working in these areas.

5.2.8 Stockpiles

Temporary topsoil and cut surplus stockpiles should be placed in an approved designated location that is relatively flat. The stockpile footprints should be stripped of vegetation and inspected by the Geotechnical Engineer prior to placement of the stockpiles, to confirm that they do not affect stability of nearby slopes. The height and side slopes should be controlled, to minimise the risk of erosion or slope instability.

The Contractor shall be responsible for the planning, management, and maintenance of stockpiles to ensure ongoing stability, appropriate drainage, and protection of surrounding ground conditions.

5.2.9 Temporary sediment retention ponds

Temporary sediment retention ponds are proposed, as detailed in the HG Earthworks drawings (Appendix H). The ponds are proposed to be located within generally flat or gently sloping parts of the Site, mainly underlain by natural ECBF terrain with localised areas of fill placed as part of the golf course development. The pond footprints should be stripped of vegetation and inspected by the Geotechnical Engineer prior to excavation, to confirm that the proposed pond details are appropriate with regard to land stability and seepage risks. Depending on the exposed ground conditions, it may be necessary to alter details of the pond such as its location, side slopes, undercutting / keying in and compaction and lining requirements.

During decommissioning, all water-softened materials at the pond bases and sides must be removed and undercut to the satisfaction of the Geotechnical Engineer. Backfilling should comply with the compaction standards for general fill (refer to Section 5.2.10) unless otherwise specified.

5.2.10 Compaction control

The Geotechnical Engineer must review and approve all fill sources to confirm their suitability and establish quality control testing requirements. Most cut material is expected to be suitable for reuse as certified engineered fill. Soils which are wet or dry of optimum may require blending with other soil materials. The earthworks contractor should allocate an on-site area for this blending process and ensure sufficient soil is retained for mixing.

Earthfill must be placed, spread, and compacted in controlled loose lifts of 200–250 mm thickness under the supervision of the Geotechnical Engineer. Suitable fill materials include granular or cohesive soils that are free of organic content and contain no particles larger than 150 mm. All earthfill placement must ensure proper bonding between successive lifts, including ripping any natural subgrade or existing fill surfaces that have dried before placing the next lift.

Earthfill must be compacted to meet or exceed the compaction control criteria, consisting of maximum allowable air voids, minimum allowable shear strengths, minimum allowable Clegg Impact Value, and minimum allowable in-situ density.

Compaction control criteria are presented in Appendix G. The criteria have been set based on the project-specific laboratory test results and are in line with those adopted for the Stage 1 consented earthworks area.

General cohesive fill specification:

Maximum Air Voids Percentage (as defined in NZS 4402:1986, for any 10 consecutive tests):

- 1 Average value not more than 10%; and
- 2 Maximum single value 12%.

Minimum Undrained Shear Strength (measured by in situ vane – IANZ calibrated, average value of any 10 consecutive tests):

- 1 Not less than 150kPa; and
- 2 Minimum single value 120kPa.

General granular fill specification:

Minimum Clegg Impact Value (measured by Clegg Impact Hammer – IANZ calibrated, average value of any 10 consecutive tests):

- 1 Average value not less than 25; and
- 2 Minimum single value 22.

Minimum Scala Penetrometer Resistance:

- 1 Average value not less than 3 blows per 100 mm; and
- 2 Minimum single value 2 blows per 100 mm.

Minimum solid density (as defined in NZS 4402:1986, for any 10 consecutive tests):

- 1 Average value not less than 82.5% of solid density (or 95% max. in-situ dry density); and
- 2 Minimum single value 80% of solid density (or 90% max. in-situ dry density).

Engineered fill testing should be performed on each 0.5 m lift as a minimum, and at the frequency recommended by NZS 4431:2022 (Ref. [25]). Alternative testing methods may be used depending on material composition.

5.3 Future building foundations**5.3.1 Preliminary shallow foundation design**

The stiff to hard ECBF soils (and the proposed engineered fill) across most of the Site are generally suitable for shallow foundations, and are expected to provide Good Ground conditions (in terms of NZS 3604:2011 (Ref. [27]) / NZS 4229:2013 (Ref. [28])) with the following exceptions:

- 1 Soils across the Site are expected to be expansive (refer to Section 5.3.2).
- 2 Preliminary analysis indicates that a 1 in 500 year earthquake could potentially result in liquefaction-induced settlements in the order of 100 to 200 mm in the CM4 (Terrace Housing) and CM5 (Hotel) areas. Refer to Section 4.2.4 of this report for more detail.

Following completion of bulk earthworks in accordance with Section 5.2, the preliminary bearing capacities provided in Table 5-1 may be adopted for the shallow foundation design within both natural cut ground (residual ECBF soils) and engineered fill areas.

Local variations in shear strength may occur within the natural cut ground, particularly where cut depths vary across building platforms. Confirmation of bearing pressures will be carried out through post-earthworks soil testing and documented in the Geotechnical Completion Report (GCR).

Table 5-1: Preliminary bearing capacities for shallow foundation design

Geological Unit	Geotechnical Ultimate Capacity (kPa)	Factored (ULS) Capacity (kPa)	Allowable (Working) Capacity (kPa)
Engineered Fill or Residual ECBF Soil	300	150	100

Where historic fill, Tauranga Group soil and colluvium are encountered, it should be verified as suitable during construction by a suitably qualified Geotechnical Engineer. Where it is inadequate, it should be undercut and replaced with engineered fill.

Based on the CPT data, preliminary analyses indicates that settlements in the order of 10 mm are expected for typical residential development supported on shallow foundations. The settlement analysis outputs are provided in Appendix F. For larger (i.e. three-storeys or greater), heavily loaded or settlement-sensitive structures, site-specific settlement analyses should be undertaken as part of the detailed design phase to confirm foundation performance under the proposed loading conditions.

Detailed design of foundation options for buildings is outside the scope of this report. If site-specific investigations and design assessment indicates that houses cannot be supported on traditional or raft shallow foundations, then piles could be considered (Section 5.3.3 of this report).

5.3.2 Expansive soils

5.3.2.1 Shallow foundations

Based on our experience in the area and supported by geotechnical laboratory testing (Section 3.7), we consider the foundation soils at the Site to lie outside the definition of “good ground” in terms of soil expansivity, as defined in NZS 3604:2011 (Ref. [27]). Soils with liquid limits (LL) exceeding 50% and linear shrinkage (LS) values greater than 15% are typically classified as expansive. The tested samples returned LL and LS values consistent with this classification.

The soils are therefore expansive, and should be designed for at least Class M (AS 2870:2011, Ref. [29]). Lot-specific testing should be carried out for buildings to confirm whether the soils are Class M or Class H. Building foundations will require specific foundation design by a suitably qualified Chartered Professional Engineer familiar with the contents of this report and responsible for design of structural elements (including foundations) of the building. Reference should be made to AS 2870:2011 (Ref. [29]) for assistance.

Factors to consider in the design include foundation depth and exposure to moisture. Foundations should be embedded into soil that will experience shrink-swell movements within the tolerances of the building. It may be possible to reduce the embedment by reducing soil moisture fluctuations beneath the foundations, using methods such as paving the zone around the building, or installing an impermeable membrane. Factors that can beneficially or detrimentally influence soil water content include:

- 1 Site paving and permeability;
- 2 Large trees close to building envelopes;
- 3 Sub-soil drains;
- 4 Garden watering;
- 5 Initial soil moisture conditions at construction time; and
- 6 Buried water storage devices, i.e. risk of future leaks.

Soils at the foundation elevation will be prone to shrinkage and swelling when exposed to changes in water content. The effects on freshly exposed material in cuts may be greater than soils that have been previously subject to seasonal cycles and this needs to be considered in design of foundations.

5.3.2.2 Trees and shrubs near proposed buildings

Trees and shrubs have high groundwater demands during the summer months. The effects of such demands on expansive soils can be substantial and potentially lead to differential building settlements well in excess of 25 mm, with resulting cracking in wall panels and floor slabs. Accordingly, it is recommended that high water demand species (such as gum, willow, cypress, etc) are not planted in the vicinity of buildings. Minimum recommended distances vary with tree species and size, and specialist advice should be obtained.

5.3.2.3 Trees and shrubs removed during site preparation

Soil moisture changes can continue for many years after trees are removed. If trees have been removed recently or will be removed as part of development, it may be necessary to specifically investigate the affected areas; and treat the ground or modify the development design.

5.3.2.4 Imported fill

The expansivity results and site classification presented in this report are applicable to the existing natural soils on this Site. If cohesive fill is imported to the Site during construction, appropriate shrink-swell testing should be undertaken on the imported fill and the site classification updated accordingly.

5.3.3 Preliminary pile foundation design

Buildings over 4-storeys high such as tall / heavy commercial / apartment buildings may require piled foundations, particularly for heavier superstructures (e.g. masonry cladding, tiled roofs, concrete structures). The geotechnical pile design parameters capacities provided in Table 5-2 may be adopted for preliminary design purposes.

Table 5-2: Preliminary capacities for pile foundation design

Geological unit	Skin Friction (kPa)		End bearing (kPa)	
	Geotechnical Ultimate Capacity	Maximum ULS Capacity	Geotechnical Ultimate Capacity	Maximum ULS Capacity
Engineered Fill	60	30	N/A	N/A
Residual ECBF Soil	40	20	N/A	N/A
Weathered ECBF	80	40	1,000	500
Moderately – Slightly Weathered ECBF Rock	120	60	5,000	2,500

Notes:

- (1) Skin friction should be ignored within the upper three pile diameters from the ground surface.
- (2) A minimum embedment of three pile diameters into the underlying competent ECBF rock is required to mobilise the full end bearing capacity.
- (3) The geotechnical ultimate capacity should be reduced by the appropriate geotechnical strength reduction factor (e.g. $\phi_g = 0.5$).

5.4 Retaining walls

Excavation and drainage for retaining walls will be determined once further detailed design work has been undertaken for any required retaining structures. Geotechnical conditions of consent are recommended to ensure that appropriate geotechnical and groundwater diversion (where required) methodologies are employed in the design of the retaining walls.

5.5 Excavations for buildings and basements

Several of the proposed buildings within the Site include basement car parks extending up to one level below ground. Construction of these basements will require earthworks and excavation. It is likely that some of the basements will also require groundwater diversion (i.e. potentially beyond the permitted activities allowed under E7.6.1.10), although it is also likely that any groundwater encountered will be perched rather than representative of the wider groundwater table. However, this will only be determined once further detailed design work is undertaken for each building.

The planning report includes groundwater diversion as a reason for consent to ensure that the appropriate consent matters are included within the application, where it eventuates that groundwater diversion is required. In addition, it is proposed that specific basement construction methodologies are required for each basement as part of the further detailed design process for each building.

Geotechnical conditions of consent are recommended to ensure that appropriate geotechnical methodologies are employed in the design and construction of the basements. Furthermore, specific groundwater diversion conditions (e.g. ground settlement monitoring and mitigation measures) are recommended to ensure the effects of groundwater diversion are appropriately managed.

General commentary on the likely geotechnical and groundwater effects associated with basement construction is provided in the following sections. Recommended conditions of consent are set out in Section 7 of this report.

5.5.1 Village Centre (VC1)

A lower-ground level carpark is proposed across much of the village centre block at VC1², as shown in Figure 5.1 and Figure 5.2.

As shown on the earthworks cross-sections 32, 33 and 34 in Appendix J (replicated in Figure 5.3), this site will generally be cut down 2 m from its existing ground level, creating a gentle slope falling towards the west. The architectural concepts presented in Figure 5.1 and Figure 5.2 show the basement subsequently formed level with the lower western side of the site at 26.5 mRL, requiring retaining walls to be built around the eastern sides. The retaining walls will be up to approximately 3.5 m high, though as shown on Figure 5.1, the highest retaining walls will be well inside the property, i.e. a substantial distance from the footpath and road on the eastern side of the block.

Geotechnical investigations were carried out in the Village Centre block in 2025 (Ref. [9][10]). As shown in the investigation logs, very stiff to hard silt/clay ECBF (HA-VC02, CPT-VC01, CPT-VC02) and locally-derived ECBF fill (HA-VC01) underlies much of the site, except at the northeastern corner near the pond where HA-X-03 encountered fill overlying stiff to very stiff silty clay alluvial deposits.

The fill is likely to be largely removed by the proposed earthworks, exposing the underlying alluvial and ECBF soils. Some of the ECBF soils at final earthworks cut level may be firm to stiff (CPT-VC01 at approximately 2 m bcgl), however these soils may strengthen as they dry out following exposure. The soils at basement excavation level (26.5 mRL) are expected to be hard (CPT-VC01 at approximately 4 m bcgl and CPT-VC02 at approximately 5.5 m bcgl).

Groundwater was encountered in HA-X-03 at 1.3 m bcgl (approx. 27.7 mRL), adjacent to the pond. Perched groundwater was encountered in HA-VC01 at 2.14 m bcgl (29.9 mRL), whereas HA-VC02 was dry upon completion of the borehole. As noted in Section 3.8.6 of this report, both historical experience and recent earthworks within Stage 1 of this site have shown that these perched groundwater tables tend to build up to 1 m to 2 m head, as they gradually seep / cascade downwards to the permanent water table closer to the base of the gullies and sea level.

The pond will be excavated and removed as part of the general earthworks in this area (to the right of cross-section 33, Appendix J), thereby removing that source of groundwater. As noted in Section 5.2.5 of this report, subsoil drains should be used to intercept groundwater seepages in low areas during bulk earthworks, which will help to mitigate against concentrated flows of perched water that could affect basement excavations.

In general, perched groundwater can be expected to be encountered in the sides of the excavation, but these water tables typically drain away within days of the excavation taking place. Longer lasting seepages should be redirected to subsoil drainage systems.

We have recommended conditions of consent to ensure the retaining wall design considers the effects of excavation and draining of perched water upon any nearby buildings or infrastructure at the time of construction. Geotechnical and groundwater effects of the basement are generally anticipated to be minor, considering that:

- 1 Draining of perched groundwater with low heads typically cause little change in effective stress, across a relatively small area;
- 2 ECBF soils are residually weathered in place, and therefore less susceptible to consolidation due to effective stress changes;
- 3 The site is surrounded by roads, so any buildings that are already in place prior to construction of the Village Centre lower ground level carpark will be a substantial distance from the excavation; and
- 4 The proposed excavation retaining walls are some distance inboard from the site boundaries.

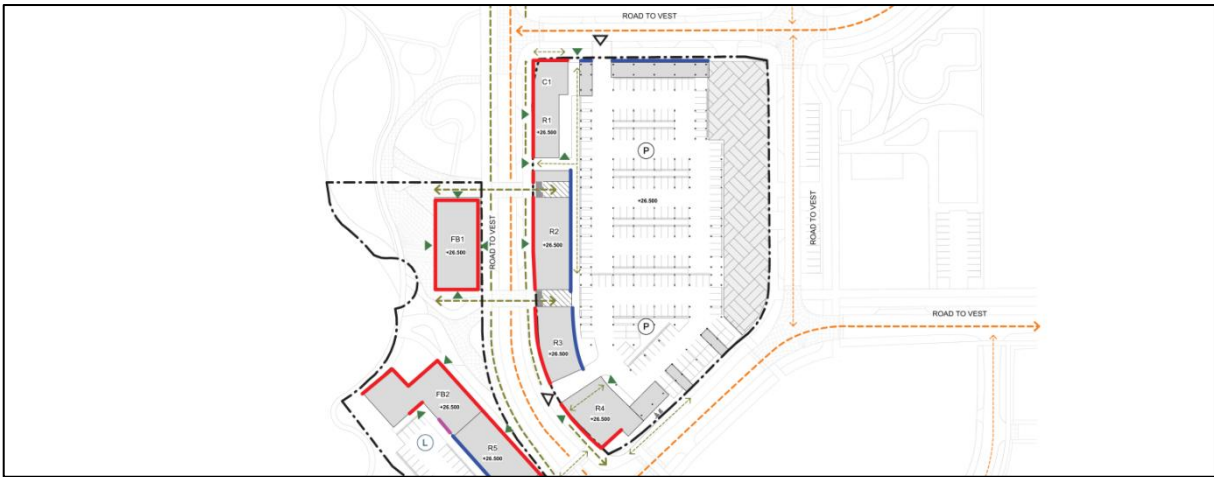


Figure 5.1: Plan, concept design for the village centre carpark (excerpt from drawing FC-VC0-20 rev A)

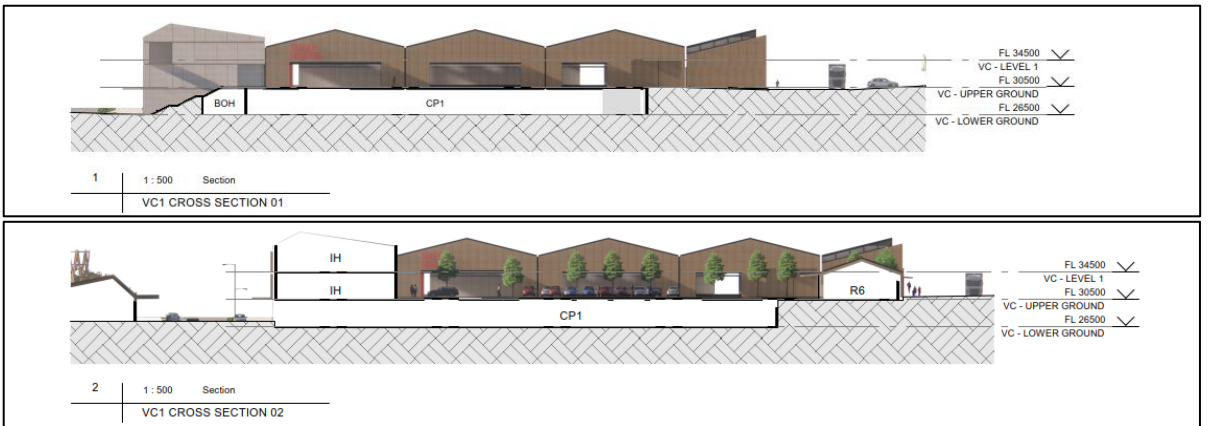


Figure 5.2: Typical sections, concept design for the village centre carpark (excerpt from drawing FC-VC1-200 rev A)

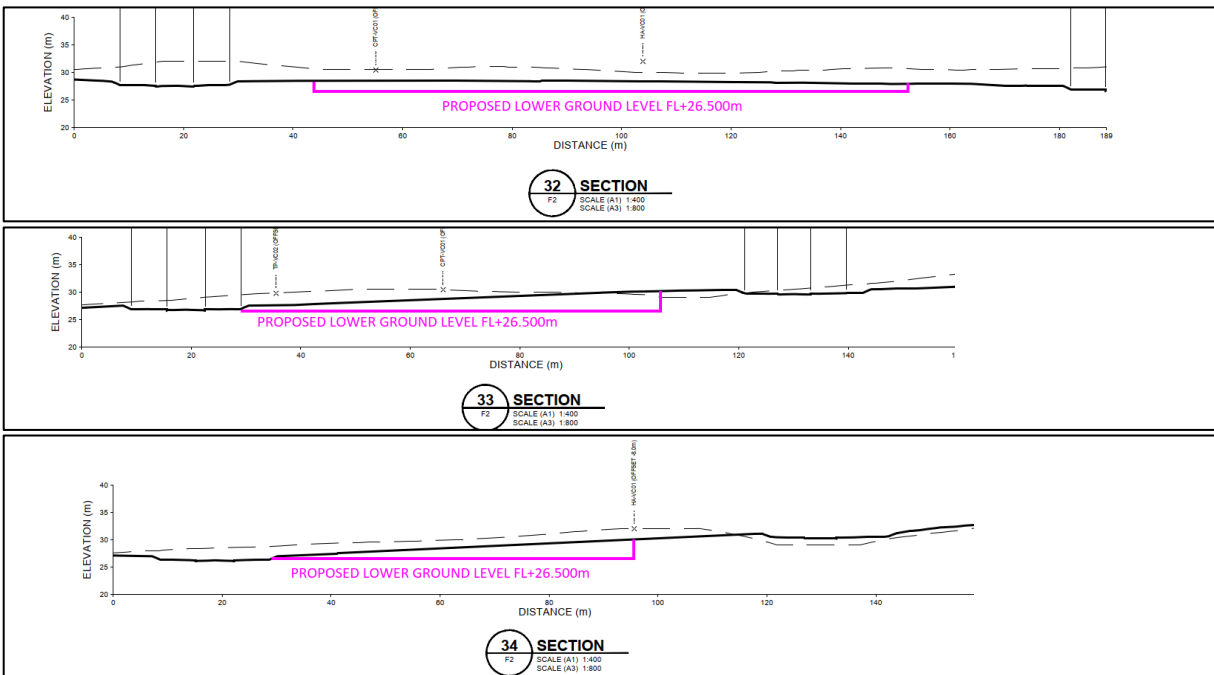


Figure 5.3: Cross-sections showing the proposed bulk earthworks cut/fill and the conceptual basement level (refer to Appendix J for the fully detailed cross-sections)

² Jasmax (2 April 2026), drawing set (49 pages), *Beachlands – Fast Track Consent: Village Centre*. Project No. 220103. Rev A, DRAFT.

5.5.2 Village Centre (VC3)

A carparking basement is proposed to service the overlying apartment buildings at VC3², as shown in Figure 5.4.

As shown on the earthworks cross-sections 28 and 29 in Appendix J (replicated in Figure 5.5), the general earthworks will build up the site with up to 2 m thickness of fill, before the basement itself is excavated back down through the new fill and down to a maximum of approximately 3 m below existing ground level.

The architectural concepts show the basement excavated along the boundary with the road, with a retaining wall of approximately 6 m height (i.e. through approximately 3 m of new fill over 3 m of existing ground).

Hand auger borehole HA-VC03 and CPT-VC04 (Ref. [9][10]) indicates that some existing fill may still be present at the proposed basement level of 23.956 mRL, comprising very stiff clayey silt derived from the underlying residual ECBF soils (very stiff to hard clayey silts and silty fine sands).

Groundwater was measured in a groundwater piezometer in HA-VC03, recovering to 1.4 m bcgl (23.6 mRL) following drilling. This water level was found to be relatively unresponsive to rainfall during monitoring from May to June 2025. As noted in Section 3.8.6 of this report, this is expected to be perched groundwater of limited pressure head, and the permanent water table is expected to be much deeper, controlled by the base of the gullies and sea level. Perched groundwater can be expected to be encountered in the sides of the excavation, but these water tables typically drain away within days of the excavation taking place. Longer lasting seepages should be redirected to subsoil drainage systems.

We have recommended conditions of consent to ensure the retaining wall design considers the effect of excavation and draining of perched water upon any nearby buildings or infrastructure at the time of construction. Geotechnical and groundwater effects of the basement are generally anticipated to be minor, considering that:

- 1 Draining of perched groundwater with low heads typically cause little change in effective stress, across a relatively small area;
- 2 ECBF soils are residually weathered in place, and therefore less susceptible to consolidation due to effective stress changes; and
- 3 The deepest retaining walls are adjacent to the road, so any buildings that are already in place prior to excavation of this basement will be a substantial distance from the excavation.

Conversely, any buried infrastructure in the footpath and road adjacent to the proposed basement should consider ground movements that could be caused by the proposed basement. Mitigation measures that could be considered include:

- 1 Construct the basement before completion of the road, footpath and buried infrastructure;
- 2 Installing buried services on the eastern side of the road, further away from the proposed basement;
- 3 Use of settlement-tolerant pipes/ducts, e.g. PVC and PE; and
- 4 Allow some margin in the gradient of gravity drains.

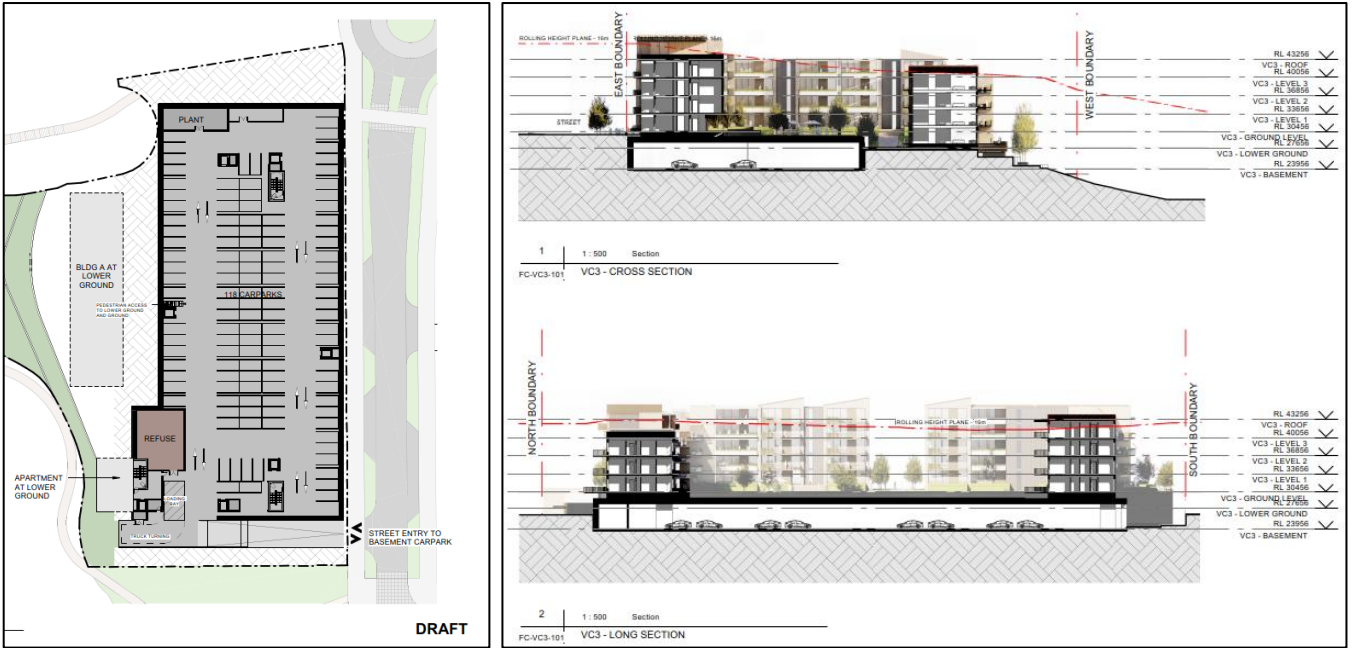


Figure 5.4: Concept plan and sections for the VC3 apartment basement (excerpt from drawings FC-VC3-104 & -200 rev A)

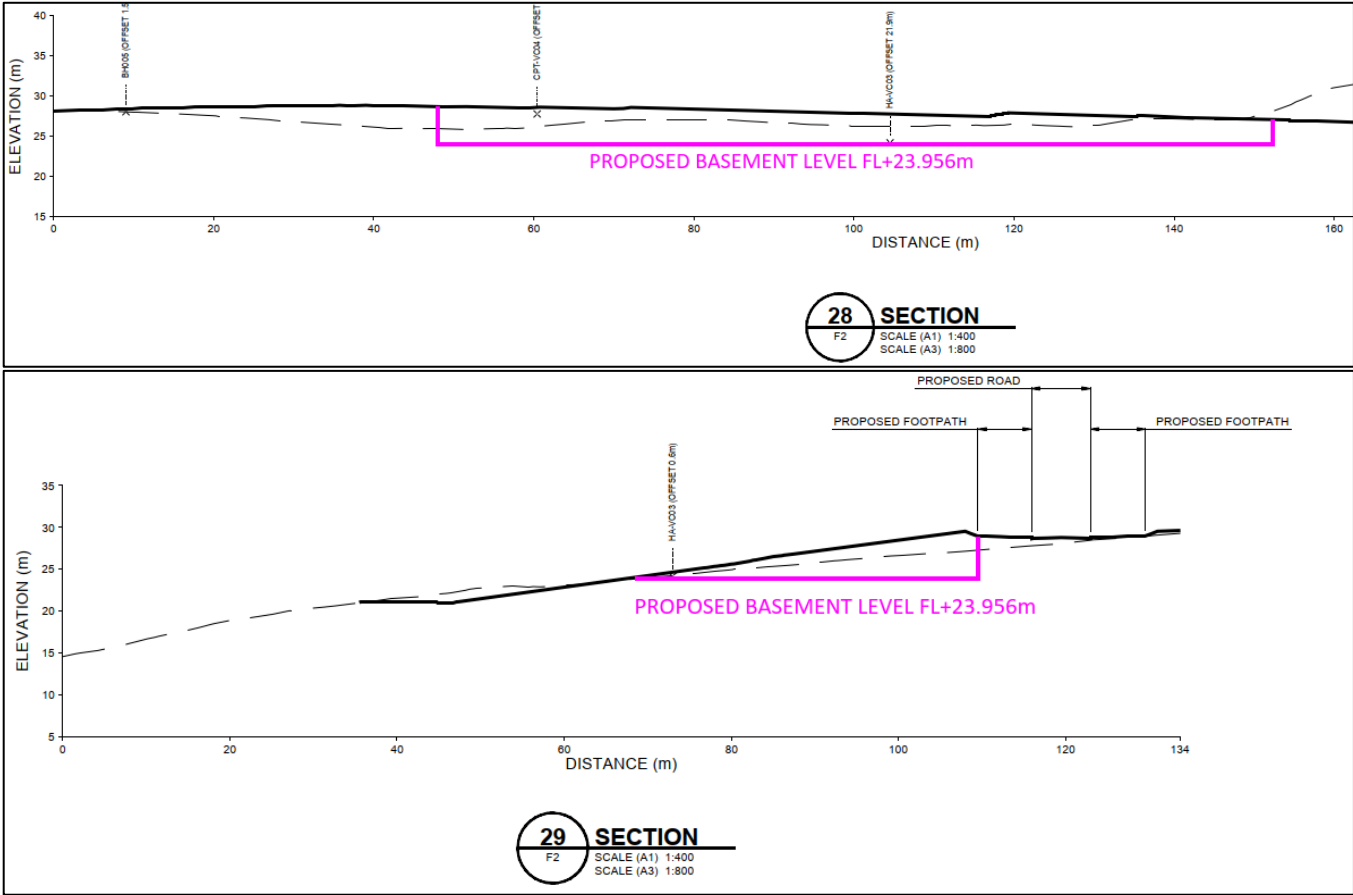


Figure 5.5: Cross-sections showing the proposed bulk earthworks cut/fill and the conceptual basement level (refer to Appendix J for the fully detailed cross-sections)

5.5.3 Village Centre (VC4)

A carparking basement is proposed to service the overlying apartment buildings at VC4², as shown in Figure 5.6.

As shown on the cross-sections 30 and 31 in Appendix J (replicated in Figure 5.7), minimal earthworks are proposed at this site. The architectural concepts indicate that the basement will be excavated up to a maximum of approximately 5.5 m depth.

The basement is proposed to extend along the boundary with the road, with a retaining wall of approximately 6 m height (i.e. through approximately 3 m of new fill over 3 m of existing ground).

Hand auger boreholes HA-VC03 and HA-VC04 and CPT-VC04 (Ref. [9][10]) indicate that the site is generally underlain by a thin veneer of topsoil and stiff to very stiff silt fill, overlying very stiff to hard residually weathered ECBF (silty clay / clayey silt). Alluvial deposits (very stiff silty clay as found in HA-X-03) are likely to be present in the southeast of the site, in the pre-existing low-lying area.

Groundwater was not encountered in HA-VC04, but was encountered adjacent to the pond in HA-X-03. As noted in Section 3.8.6 of this report, shallow groundwater is expected to be perched with limited pressure head, and the permanent water table is expected to be much deeper, controlled by the base of the gullies and sea level.

The pond to the southeast of the site will be excavated and removed as part of the general earthworks in this area, thereby removing that source of groundwater. As noted in Section 5.2.5 of this report, subsoil drains should be used to intercept groundwater seepages in low areas during bulk earthworks, which will help to mitigate against concentrated flows of perched water that could affect basement excavations.

In general, perched groundwater can be expected to be encountered in the sides of the excavation, but these water tables typically drain away within days of the excavation taking place. Longer lasting seepages should be redirected to subsoil drainage systems.

We have recommended conditions of consent to ensure the retaining wall design considers the effect of excavation and draining of perched water upon any nearby buildings or infrastructure at the time of construction. Geotechnical and groundwater effects of the basement are generally anticipated to be minor, considering that:

- 1 Draining of perched groundwater with low heads typically cause little change in effective stress, across a relatively small area;
- 2 ECBF soils are residually weathered in place, and therefore less susceptible to consolidation due to effective stress changes;
- 3 The site is surrounded by roads, so any buildings that are already in place prior to construction of the basement will be a substantial distance from the excavation; and
- 4 The proposed excavation retaining walls are some distance inboard from the site boundaries.



Figure 5.6: Concept plan and sections for the VC4 apartment basement (excerpt from drawings FC-VC4-105 & -200 rev A)

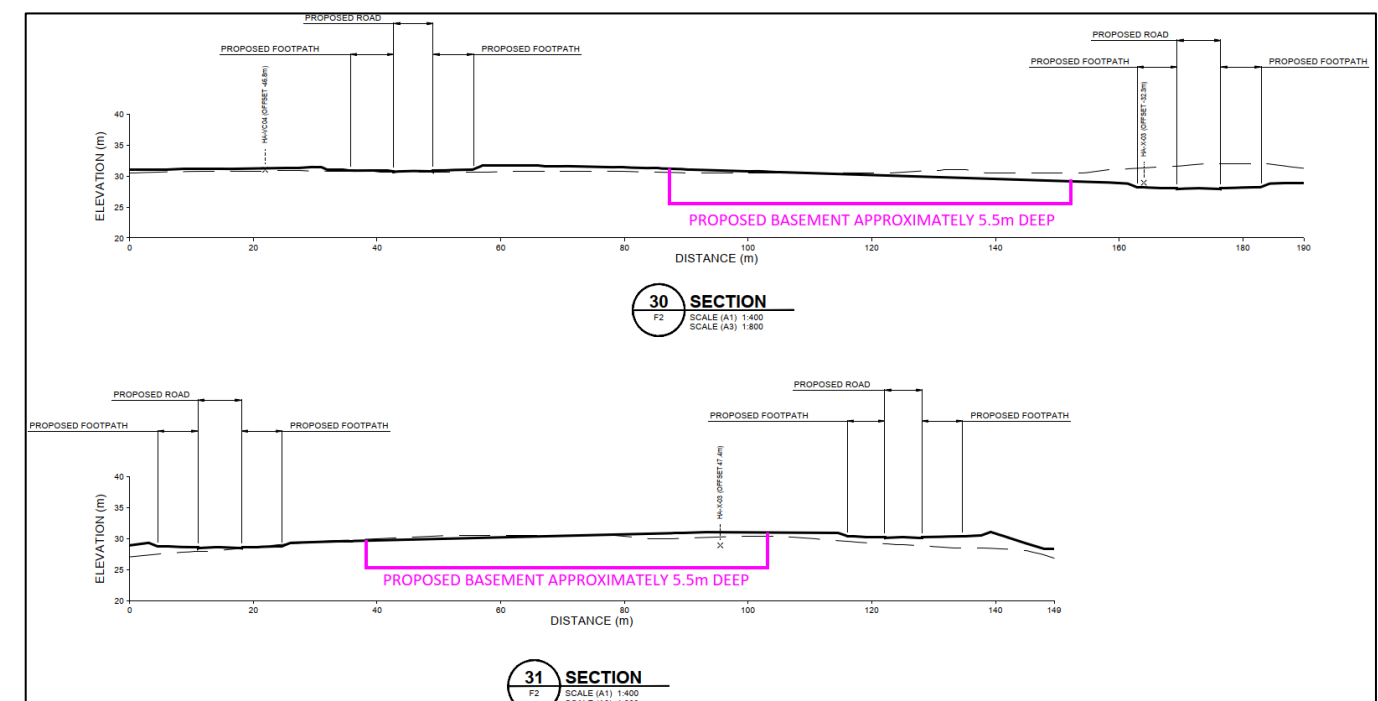


Figure 5.7: Cross sections showing the proposed bulk earthworks cut/fill and the conceptual basement level (refer to Appendix J for the fully detailed cross sections)

5.5.4 Community Precinct (CM3)

A carparking basement is proposed to service the overlying apartment buildings at CM3³, as shown in Figure 5.8.

Hand auger boreholes BH016 and BH017 (Appendix B) indicate that the site is generally underlain by stiff to hard residually weathered ECBF (silty clay / clayey silt), transitioning to silty fine sand to sand below approximately 30.4 mRL to 32.0 mRL. CPT-VC02 shows a steady increase in strength below 3 m depth, becoming hard / medium dense below 4 m depth, and very dense below 5 m bcgl.

As shown on the cross-sections 36 and 37 in Appendix J (replicated in Figure 5.9), bulk earthworks cut is proposed across this site, creating a general crossfall towards the west. The architectural concepts indicate that the basement will be excavated up to a maximum of approximately 4.5 m depth below the finished ground level (up to approximately 7 m below existing ground level). The basement is therefore likely to expose very stiff to hard silts/clays and medium-dense to dense silty sands.

Groundwater was encountered in both boreholes. As noted in Section 3.8.6 of this report, shallow groundwater is expected to be perched with limited pressure head, and the permanent water table is expected to be much deeper, controlled by the base of the gullies and sea level. As noted in Section 5.2.5 of this report, subsoil drains should be used to intercept groundwater seepages in low areas and/or sandy soil lenses during bulk earthworks, which will help to mitigate against concentrated flows of perched water that could affect basement excavations.

In general, perched groundwater can be expected to be encountered in the sides of the excavation, but these water tables typically drain away within days of the excavation taking place. Longer lasting seepages should be redirected to subsoil drainage systems.

We have recommended conditions of consent to ensure the retaining wall design considers the effect of excavation and draining of perched water upon any nearby buildings or infrastructure at the time of construction. Geotechnical and groundwater effects of the basement are generally anticipated to be minor, considering that:

- 1 Draining of perched groundwater with low heads typically cause little change in effective stress, across a relatively small area;
- 2 ECBF soils are residually weathered in place, and therefore less susceptible to consolidation due to effective stress changes;
- 3 The site is surrounded by roads, so any buildings that are already in place prior to construction of the basement will be a substantial distance from the excavation; and
- 4 The deepest parts of the proposed excavation are a significant distance from any proposed nearby buildings.



Figure 5.8: Concept plan and sections for the CM3 apartment basement (excerpt from drawings FC-CM3-105 & -200 rev A)

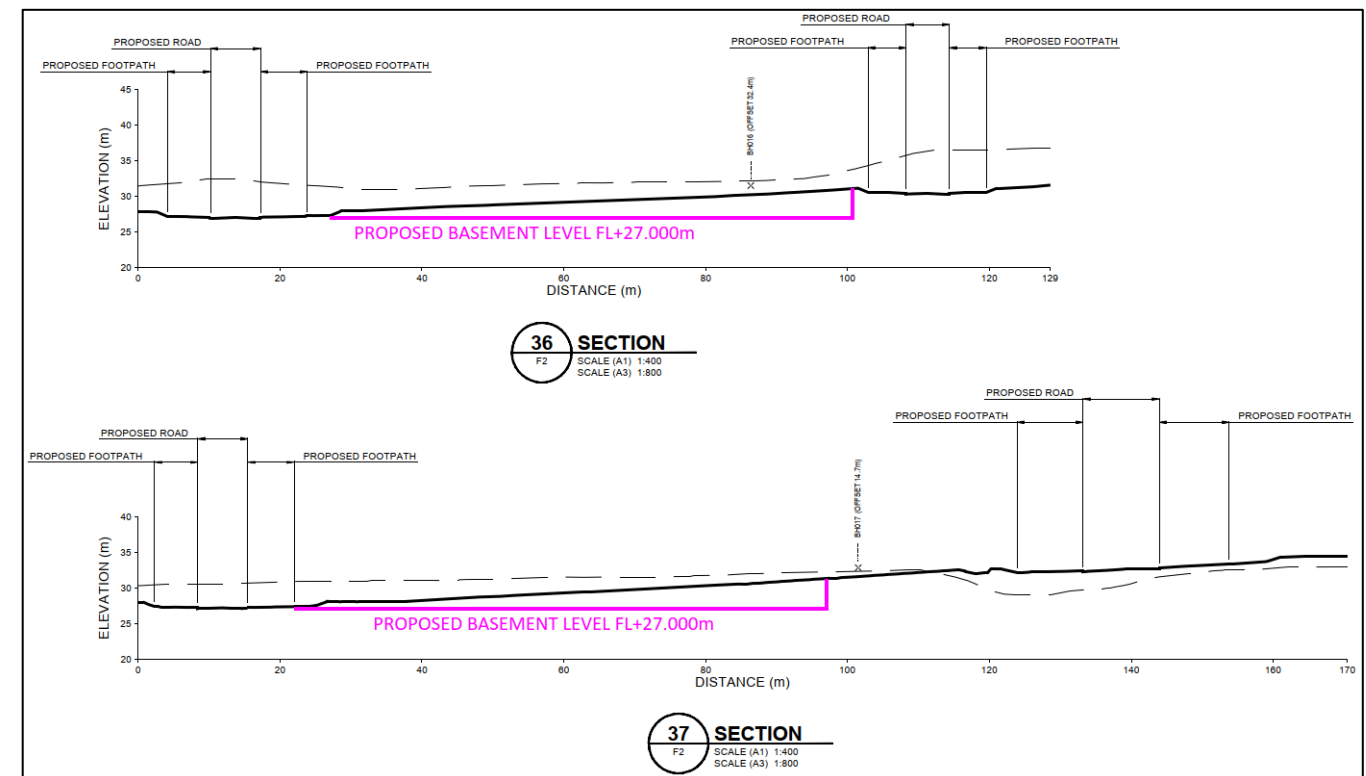


Figure 5.9: Cross sections showing the proposed bulk earthworks cut/fill and the conceptual basement level (refer to Appendix J for the fully detailed cross sections)

³ Jasmax (2 April 2026), drawing set (33 pages), *Beachlands – Fast Track Consent: Community Precinct*. Project No. 220103. Rev A, DRAFT

5.5.5 Hotel, Community Precinct (CM5)

A hotel³ is proposed to be built as an extension of the existing clubhouse building, with a basement carpark, as shown in Figure 5.10. This area is currently beneath a pond, and bulk earthworks plans for this area propose general fill (Figure 5.11). The proposed basement will be constructed at FL 34.200 m, approximately 2 m below the estimated level of the existing pond bed (Figure 5.12).

Hand auger borehole HA-X-05 was put down at the south of the existing pond (Figure 5.11), encountering stiff fill (silty fine to medium sand) over very stiff alluvial deposits (silty clay). CPT-014 to the north of the pond and the proposed hotel encountered stiff sandy silts and clays down to approximately 6 m depth, becoming very stiff below approximately 8 m depth, and hard below approximately 10 m depth. Groundwater is currently governed by the pond, which is inferred to create a general perched water table surrounding the pond, overlying a deeper regional groundwater table at the level of the surrounding gullies and sea level.

Based on the investigations, the basement is likely to be founded in stiff to very stiff alluvial soils, across the centre of the basement, potentially transitioning to ECBF soils at the north and south; reflecting the original gully geology prior to landscaping for the golf course. Bulk earthworks will need to muck out any soft sediments in the base of the pond prior to placement of general fill. As noted in Section 5.2.5 of this report, subsoil drains should be used to intercept groundwater seepages in low areas and/or sandy soil lenses during bulk earthworks, which will help to mitigate against concentrated flows of perched water that could affect basement excavations. Subsoil drainage layout should be considered against the proposal for a permanent pond to the west of the new hotel (labelled “A” in Figure 5.10).

In general, perched groundwater can be expected to be encountered in the sides of the basement excavation (particularly where the basement crosses the historical gully). Longer lasting seepages should be redirected to subsoil drainage systems. Tanking of the basement should be considered, where it is proposed to be lower than the adjacent pond. Conversely, there may be cost and maintenance benefits in coordination of the pond level and general landscaping design with basement tanking or moisture control.

We have recommended conditions of consent to ensure the retaining wall design considers the effect of excavation and draining of perched water upon nearby buildings or infrastructure at the time of construction. Care will be required to design the excavation and construction sequence near the existing clubhouse building, to manage settlement effects due to groundwater pressure changes and movement of the soils due to basement excavation and retention.

The proposed basement is a significant distance away from other buildings and nearby roads, and geotechnical and groundwater effects on structures/infrastructure apart from the existing clubhouse are generally expected to be minor. This should be assessed during design, prior to building consent and engineering plan approval application.



Figure 5.10: Concept plan for the CM5 hotel basement (excerpt from drawings FC-CM5-100 & -105 rev A)



Figure 5.11: Site investigation location plans, and cross-section 38 location. Refer to Appendix J for the full plan

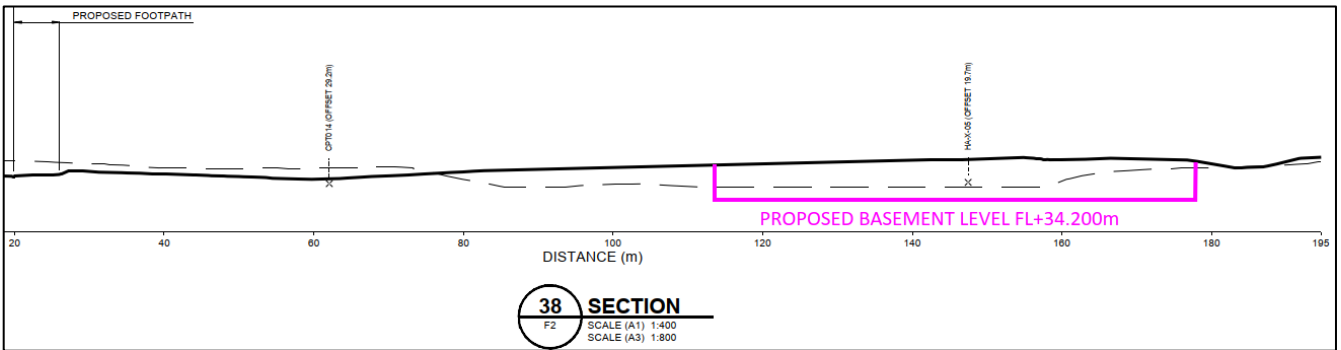


Figure 5.12: Cross-section showing the proposed bulk earthworks cut/fill and the conceptual basement level (refer to Appendix J for the fully detailed cross-sections)

5.5.6 Apartments in the Marina Point and Coastal Precincts

Several apartment buildings are proposed for the Marina Point and Coastal Precincts, as shown in Figure 5.13 and Figure 5.14. Apartment Types 1 to 5 propose single-level or partial basements, whereas the smaller Type 6 apartments appear unlikely to require basement excavations (Figure 5.15).

Ground conditions in these precincts are described in Section 3.8 of this report.

As noted in Section 3.8.6 of this report, both historical experience and recent earthworks within Stage 1 of this site have shown that perched groundwater in this terrain tends to build up to 1 m to 2 m head, as it gradually seeps / cascades downwards to the permanent water table closer to the base of the gullies and sea level.

The cross-sections in Appendix J show the proposed general bulk earthworks in these areas. Cuts of up to 5 m depth are proposed along the ridge of Marina Point (cross-sections 22, 26) to create fill for placement further to the east. Otherwise, earthworks comprise minor smoothing of the existing contours (cross-sections 21, 23, 24, 25, 27, 39, 40). The finished ground level at the apartments closer to the community centre (cross-section 35) will be approximately 2 m to 3 m below existing ground level, while the southernmost apartment (cross-section 41) might straddle localised cut-to-fill of up to 3 m depth. As noted in Section 5.2.5 of this report, subsoil drains should be used to intercept groundwater seepages in low areas and areas of cut during bulk earthworks, which will help to mitigate against concentrated flows of perched water that could affect basement excavations.

Broadly speaking, the basements of up to 3.5 m depth will be constructed within very stiff ECBF or fill derived from ECBF, typically comprising sandy to clayey silts. In general, perched groundwater can be expected to be encountered in the sides of basement excavations, but these water tables typically drain away within days of the excavation taking place. Longer lasting seepages should be redirected to subsoil drainage systems.

We have recommended conditions of consent to ensure the retaining wall design considers the effect of excavation and draining of perched water upon any nearby buildings or infrastructure at the time of construction. In general, effects are likely to be minor, except if excavations are close to existing infrastructure or buildings. Geotechnical and groundwater effects of the basements will need to be assessed at the time of detailed design and prior to application for building consent and engineering plan approval.



Figure 5.13: Conceptual locations of apartment buildings in the Marina Point Precinct. Excerpt from drawing 4655-WD.103⁴



Figure 5.14: Conceptual locations of apartment buildings in the Coastal Precinct. Excerpt from drawing 4655-WD.103⁴

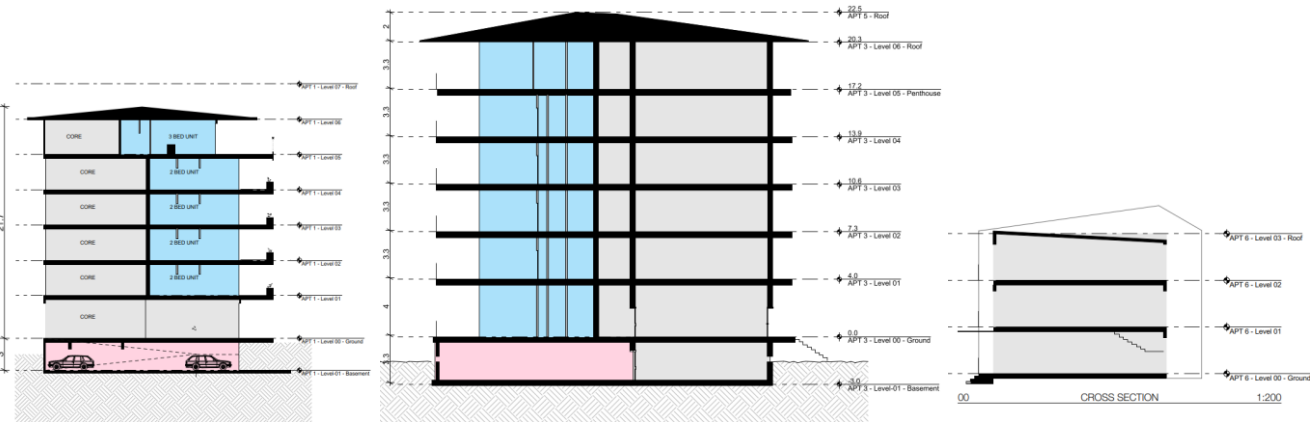


Figure 5.15: Conceptual cross-sections through typical apartments. Excerpt from drawings 4655-RC_1.05, -RC_3.05 & -RC_6.03⁴

⁴ Studio Pacific Architecture (02/04/2026), drawing set (60 pages), *Beachlands Apartments Fast Track Consent*. Job No. 4655. Rev 01-WIP, DRAFT.

5.6 Future Urban Zone

The properties at 620, 712, 758 and 770 Whitford-Maraetai Road (and a small part of the property at 110 Jack Lachlan Drive) are zoned FUZ. Limited works are proposed within the FUZ, as shown Figure 5.16. Some geotechnical inputs will be required at detailed design stage as described below:

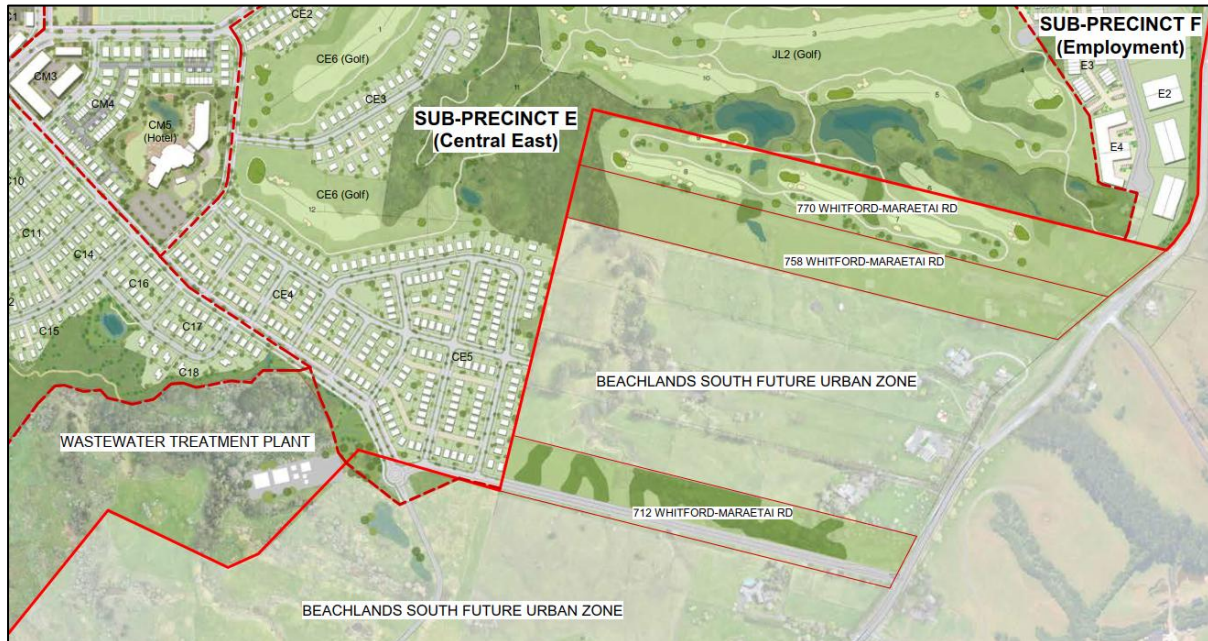


Figure 5.16: Limited works proposed within the FUZ (golf fairways in 758 & 770 Whitford Maraetai Rd, access road in 712 Whitford-Maraetai Rd, and the new Wastewater Treatment Plant).

5.6.1 Wastewater effluent dripper irrigation field

The Wastewater Concept Design and AEE report (Ref.[30]) shows that effluent will be disposed to land using a dripper irrigation field to the south of the new Wastewater Treatment Plant, at an estimated rate of up to 3 mm/day. The exact location of the field will be confirmed during detailed design. The geotechnical stability of the dripper irrigation field should be assessed during detailed design (we recommend that this should be a condition of consent, as listed in section 7.1 of this report).

5.6.2 Access road on 712 Whitford-Maraetai Road

An access road will be constructed on the 712 Whitford-Maraetai Road property, as shown in Figure 5.16. This access road will require geotechnical input at the design stage, and conditions of consent are recommended to manage any geotechnical effects associated with the design and construction of this road (listed in section 7.1 of this report).

5.6.3 New golf fairways on 758 and 770 Whitford-Maraetai Road

New golf fairways are proposed at the above properties. Landscaping for the fairways may involve minor earthworks (typically less than 3m cut or fill). Consent conditions are recommended to ensure that appropriate geotechnical methodologies are employed in the design and construction of these earthworks.

6 Safety in design

The proposed design landform requires site excavations that may include geotechnical works such as undercuts and temporary excavations as specified in this report. These works can potentially present significant health and safety risks to contractors, site personnel, and inspectors during construction.

In conducting our scope of work, we have considered and incorporated Safety in Design (SiD) principles in accordance with the Auckland Code of Practice for Land Development and Subdivision (Ref. [5]). Our SiD considerations have been applied to those aspects of the project for which we are responsible, and are based on our understanding of the proposed design and construction methodology at the time of reporting.

SiD must consider the construction, operation, maintenance, and ultimate demolition phases of the relevant works. Our design has specifically aimed to eliminate or mitigate foreseeable risks, particularly in relation to geotechnical features such as excavations and subsoil drainage network.

It is important to note that the focus of this report is on design-phase risk management. While we have identified key hazards and implemented design measures to reduce risk, the Contractor (as the Person Conducting a Business or Undertaking, PCBU), is responsible for assessing and managing all construction-related health and safety risks on site. This includes conducting detailed task-specific risk assessments and implementing appropriate controls in line with the Construction Health and Safety requirements.

Specifically:

- 1 T+T has designed undercuts and drainage systems such that no personnel are required to enter unbattered or unsupported excavations at any stage during construction.
- 2 If a contractor considers that an excavation or design element cannot be constructed safely using the specified method, T+T must be consulted immediately to review the design and agree on an alternative safe construction methodology.

Our formal SiD Risk Assessment is included in Appendix K of this report. This document must be communicated to all affected parties, including designers, contractors, subcontractors, and site supervisors, and integrated into the site-specific risk management plans.

7 Recommended conditions of consent and further work

Recommendations for future geotechnical work as this Project proceeds through resource consent, Engineering Plan Approval, building consent and construction certification are presented below.

7.1 Recommended resource consent conditions

Geotechnical resource consent conditions recommended within this report are summarised in Table 7-1.

Table 7-1: Recommended geotechnical resource consent conditions

Recommended resource consent conditions	Background
Ground conditions should be inspected and tested by the Geotechnical Engineer as they are exposed during earthworks construction, to confirm that they are compatible with the assumed ground model.	This is a typical consent requirement, and is required for completion reporting.
Geotechnical stabilisation near the southern cliffs (shown below) should be confirmed at detailed design stage, to meet the Auckland Code of Practice for Land Development and Subdivision (Ref. [5]) requirements for slope stability. 	Preliminary assessment indicates that slope stabilisation may be required in parts of this area (as detailed in Section 4.3.7 of this report). Stabilisation measures could comprise in-ground palisade piled walls or similar, potentially combined with earthworks and/or building setback lines. Supplementary site investigations including cored machine boreholes and measurement of perched groundwater would be required for design refinement.
Geotechnical conditions of consent are recommended to ensure that appropriate geotechnical methodologies are employed in the design and construction of building basements. Furthermore, specific groundwater diversion conditions (e.g. ground settlement monitoring and mitigation measures) are recommended to ensure the effects of groundwater diversion are appropriately managed.	Several building basements are proposed (as discussed in Section 5.5 of this report). Building-specific geotechnical investigations, design and assessment of effects should be carried out as part of design development (and prior to building consent).
Excavation and drainage for retaining walls will be determined once further detailed design work has been undertaken for any required retaining structures. Geotechnical conditions of consent are recommended to ensure that appropriate geotechnical and groundwater diversion (where required) methodologies are employed in the design of the retaining walls.	
Geotechnical stability of the wastewater dripper irrigation field should be assessed during detailed design.	The dripper field will be located at the north of the 620 Whitford-Maraetai Road property (within the FUZ), but the exact location of the field has not been confirmed yet.
Geotechnical input is required at the design stage for the proposed access road at 712 Whitford-Maraetai Road. Geotechnical conditions of consent are recommended to manage the geotechnical effects associated with the design and construction of these works.	

Recommended resource consent conditions	Background
Geotechnical input is required at the design stage for the new golf fairways in 758 and 770 Whitford-Maraetai Road. Geotechnical conditions of consent are recommended to manage the geotechnical effects associated with the design and construction of these works.	

7.2 Settlement due to earthworks

Preliminary analyses indicate that consolidation settlements due to the proposed earthworks are likely to be less than 10 mm, and likely to occur relatively rapidly during placement of engineered fill. If the design is changed such that deeper fills are proposed, or if thicker or softer soils are encountered beneath proposed fills, then survey monitoring should be considered (as detailed in Section 4.6 of this report).

7.3 Confirmation of ground conditions during earthworks construction

Recommendations for earthworks are provided in Section 5.2 of this report. During excavation and construction, the Site should be examined by a suitably qualified Geotechnical Engineer competent to judge whether the exposed subsoils are compatible with the inferred conditions on which the report has been based. This is a typical condition of Building Consent and is required for Completion Reporting as required by the Auckland Code of Practice (Ref. [5]). It is important that we be contacted if there is any variation in subsoil conditions from those described in the report. Particular areas of interest (such as historical fill, slopes, and gullies) are listed in Section 4.5 and Table 4-4 (Section 4.3.7) of this report.

7.4 Supplementary site investigations for buildings

Supplementary geotechnical investigations may be needed for detailed design of buildings prior to application for building consent. The need for and quantum of investigations will depend on the scale of the buildings, and exposure to and consequence of potential geotechnical hazards.

As a normal part of building foundation design, the potential effect of expansive soils should be considered, particularly for shallow foundations. Preliminary assessment indicates that foundations should be designed for at least Class M (AS 2870:2011, Ref. [29]), and building-specific soil testing should be carried out to check whether the site soils fall into Class H.

One site investigation CPT test within the CM4 (terrace housing) and CM5 (hotel) areas in the Community Precinct indicated the potential for liquefaction. It is suspected that this result is conservative, and supplementary investigations are recommended for this location to confirm the liquefaction risk (as detailed in Section 4.2.4 of this report).

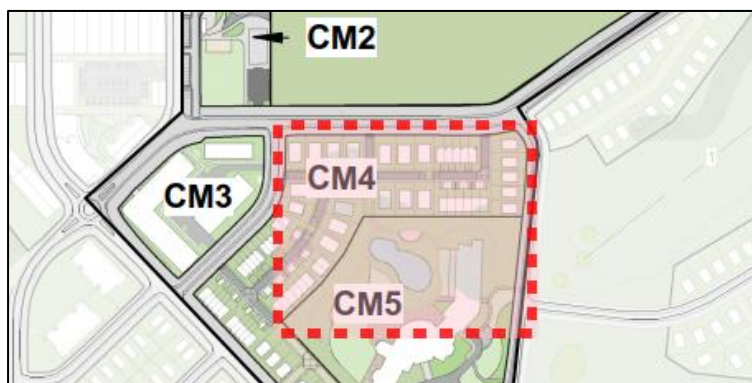


Figure 7.1: Supplementary site investigations are recommended in the CM4/CM5 area to confirm liquefaction risk.

8 Applicability

This report has been prepared for the exclusive use of our client BSLP, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

We understand and agree that our client will submit this report as part of an application under the Fast-track Approvals Act 2024 and that an Expert Panel will use this report for the purpose of assessing that application. We understand and agree that this report will be used by the Expert Panel in undertaking its regulatory functions.

Tonkin & Taylor Ltd
Environmental and Engineering Consultants

Authorised for Tonkin & Taylor Ltd by:



Mark Thomas
Project Director

Report prepared and reviewed by:

Scott Honiss
Jacob Simpson
James Lee

Engineering Geologist
Senior Geotechnical Engineer
Senior Geotechnical Engineer

25-Jun-26

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9 References

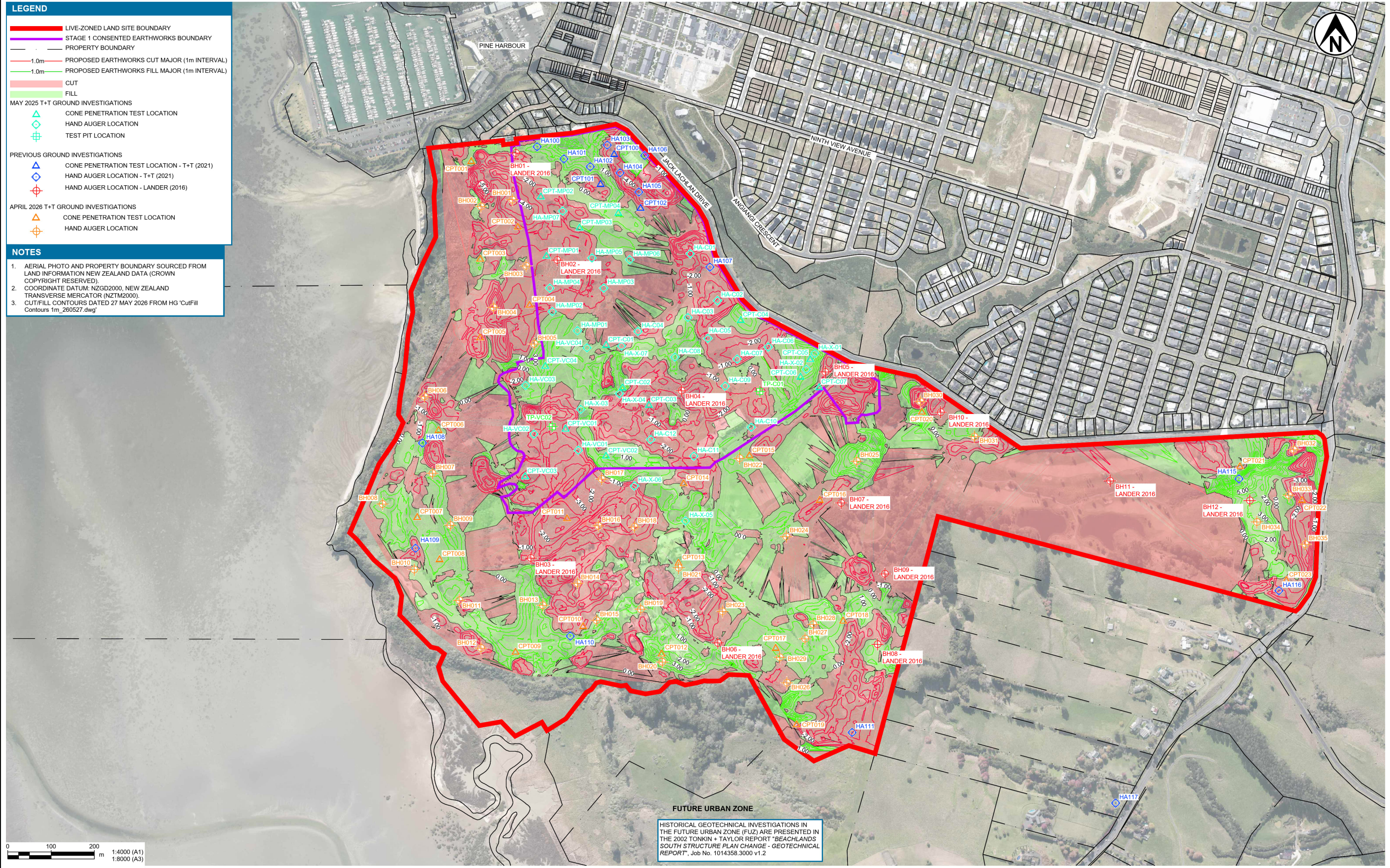
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- [5] Auckland Council, May 2023. *The Auckland Code of Practice for Land Development and Subdivision, Chapter 2: Earthworks and Geotechnical*, Version 2.0
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- [7] Lander, S.G. (dated 10 February 2016). *STATEMENT OF EVIDENCE BY SHANE GARETH LANDER ON BEHALF OF 110 FORMOSA (NZ) LIMITED, 110A JACK LACHLAN DRIVE, BEACHLANDS, For the Hearing for Submission 7318 for 081 – Rezoning and Precincts (Geographic Areas)*
- [8] Tonkin & Taylor Limited (T+T Ref: 1014358.3000.v1.2, dated November 2022). *Beachlands South Structure Plan Change, Geotechnical Report*
- [9] Tonkin & Taylor Limited (T+T Ref: 1014358.3100.v1.1, dated July 2025). *Beachlands South - Stage 1 Earthworks Consent, Geotechnical Investigation Report*
- [10] Tonkin & Taylor Limited (T+T Ref: 1014358.3100-v1.2, dated 23 September 2025). *Beachlands South Stage 1, Supplementary geotechnical investigations*
- [11] Resource Management Act (1991)
- [12] <https://nshm.gns.cri.nz/>
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- [15] Ministry of Business, Innovation & Employment (November 2021). *Earthquake geotechnical engineering practice – Module 1. Overview of the guidelines*
- [16] Australian/New Zealand Standard (2002). *AS/NZS 1170.0:2002. Structural design actions. Part 0: General principles*
- [17] Bray J.D. and Sancio R.B. (2006). "Assessment of the liquefaction susceptibility of fine-graded soils", *Journal of Geotechnical and Geoenvironmental Engineering*, 132 (9), 1165–1177
- [18] Boulanger R.W. and Idriss I.M. (2006). "Liquefaction Susceptibility Criteria for Silts and Clays", *Journal of Geotechnical and Geoenvironmental Engineering*, 132 (11), 1412–1426
- [19] Boulanger, R., & Idriss, I. (2014). *CPT and SPT based liquefaction triggering procedures*, Report No UDC/CGM-14/01. Davis: University of California
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- [21] Tonkin & Taylor Limited (T+T Ref: 1014358.2501, dated June 2025). *Beachlands South Fast Track Application - Coastal Hazard NPS-NH and AUP Risk Assessment*.
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- [24] New Zealand Geotechnical Society (NZGS), October 2024. *Slope Stability Geotechnical Guidance Series Unit 1 – General Guidance.*
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- [26] Auckland Council, June 2016. *Erosion and Sediment Control Guide for Land Disturbing Activities in the Auckland Region (GD05).*
- [27] New Zealand Standards, 2011. *NZS 3604:2011 Timber-Framed Buildings.*
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- [29] Standards Australia, 2011. *AS 2870:2011 Residential Slabs and Footings – Construction.*
- [30] GWE Consulting Engineers (June 2026). *Beachlands South Fast-track Substantive Application: Wastewater Concept Design and AEE.* V1, Job No J3021-2.

Appendix A T+T Figures

• 1014358.3200–Figure 1 [Rev 1]	Site Investigation Plan
• 1014358.3200–Figure 2 [Rev 1]	Proposed Cut/Fill Plan
• 1014358.3200–Figure 3 [Rev 1]	Geological Cross-Sections 1 & 2
• 1014358.3200–Figure 4 [Rev 1]	Geological Cross-Sections 3 & 4
• 1014358.3200–Figure 5 [Rev 1]	Geological Cross-Section 6
• 1014358.3200–Figure 6 [Rev 1]	Geological Cross-Sections 7 & 8
• 1014358.3200–Figure 7 [Rev 1]	Geological Cross-Section 9
• 1014358.3200–Figure 8 [Rev 1]	Geological Cross-Section 11 & 12
• 1014358.3200–Figure 9 [Rev 1]	Geological Cross-Section 15
• 1014358.3200–Figure 10 [Rev 1]	Geological Cross-Section 17 & 18
• 1014358.3200–Figure 11 [Rev 1]	Geological Cross-Section 19 & 20





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FIGURE CHECKED	JC	04.06.26	
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PROJECT	BEACHLANDS SOUTH BALANCE LIVE
TITLE	PROPOSED EARTHWORKS CUT/FILL PLAN
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FIG No.	FIGURE 2
REV	1

NOTES

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LEGEND

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PROPOSED GROUND

INFERRED GEOLOGICAL BOUNDARY

INFERRED GROUNDWATER LEVEL

PROPOSED FILL

PROPOSED CUT

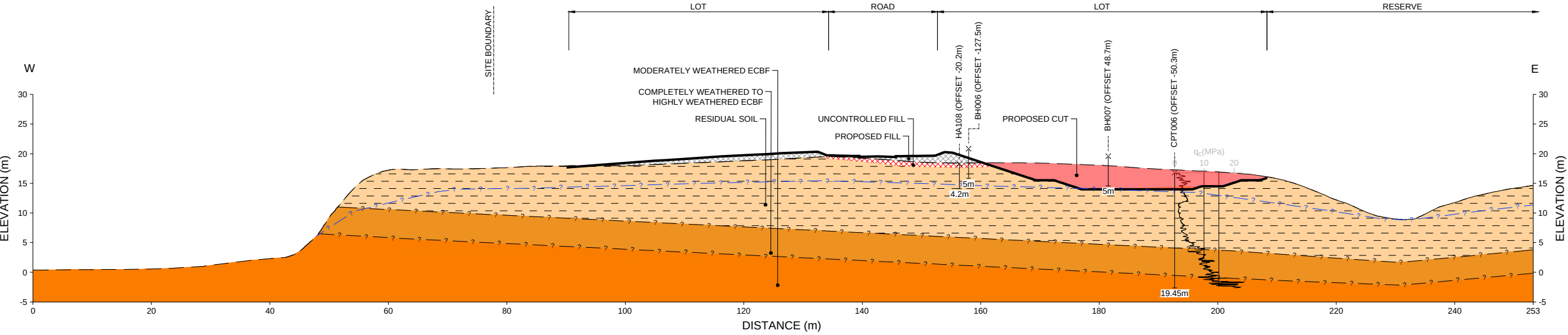
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ALLUVIUM

RESIDUAL SOIL

CW-HW ECBF

MW-SW ECBF

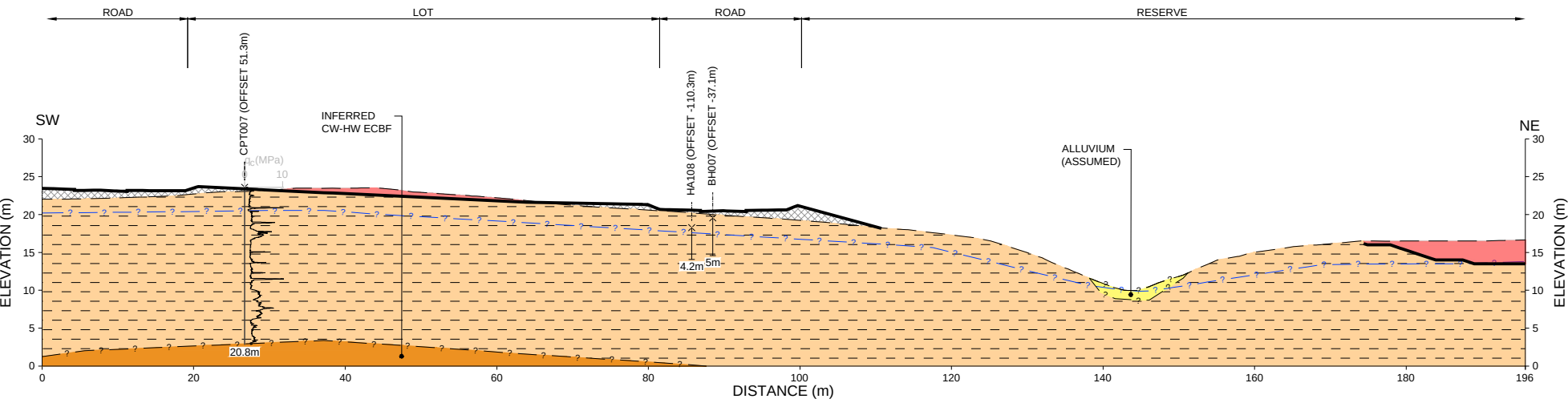


1 SECTION

F1

SCALE (A1) 1:400

SCALE (A3) 1:800



2 SECTION

F1

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SCALE (A3) 1:800



1:400 (A1)

1:800 (A3)



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CLIENT

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PROJECT

BEACHLANDS SOUTH BALANCE LIVE-ZONE

TITLE

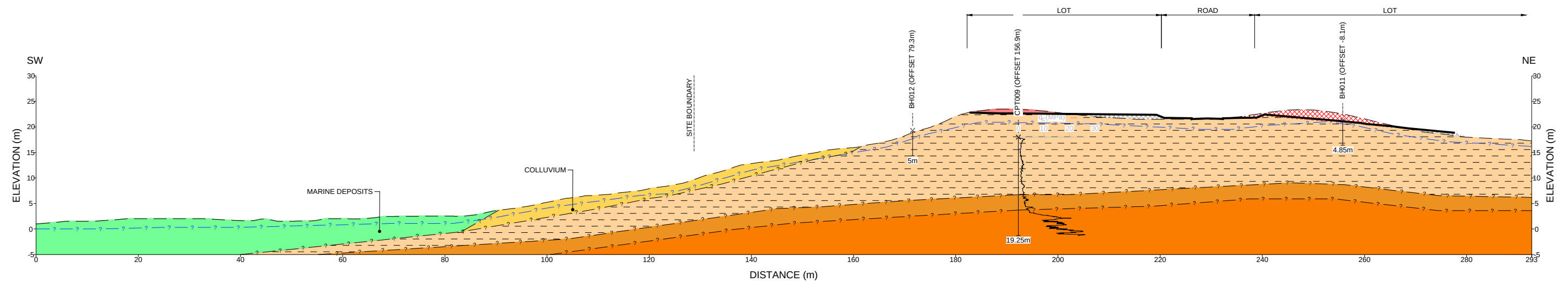
GEOLOGICAL CROSS-SECTIONS 1 & 2

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FIG No. FIGURE 3

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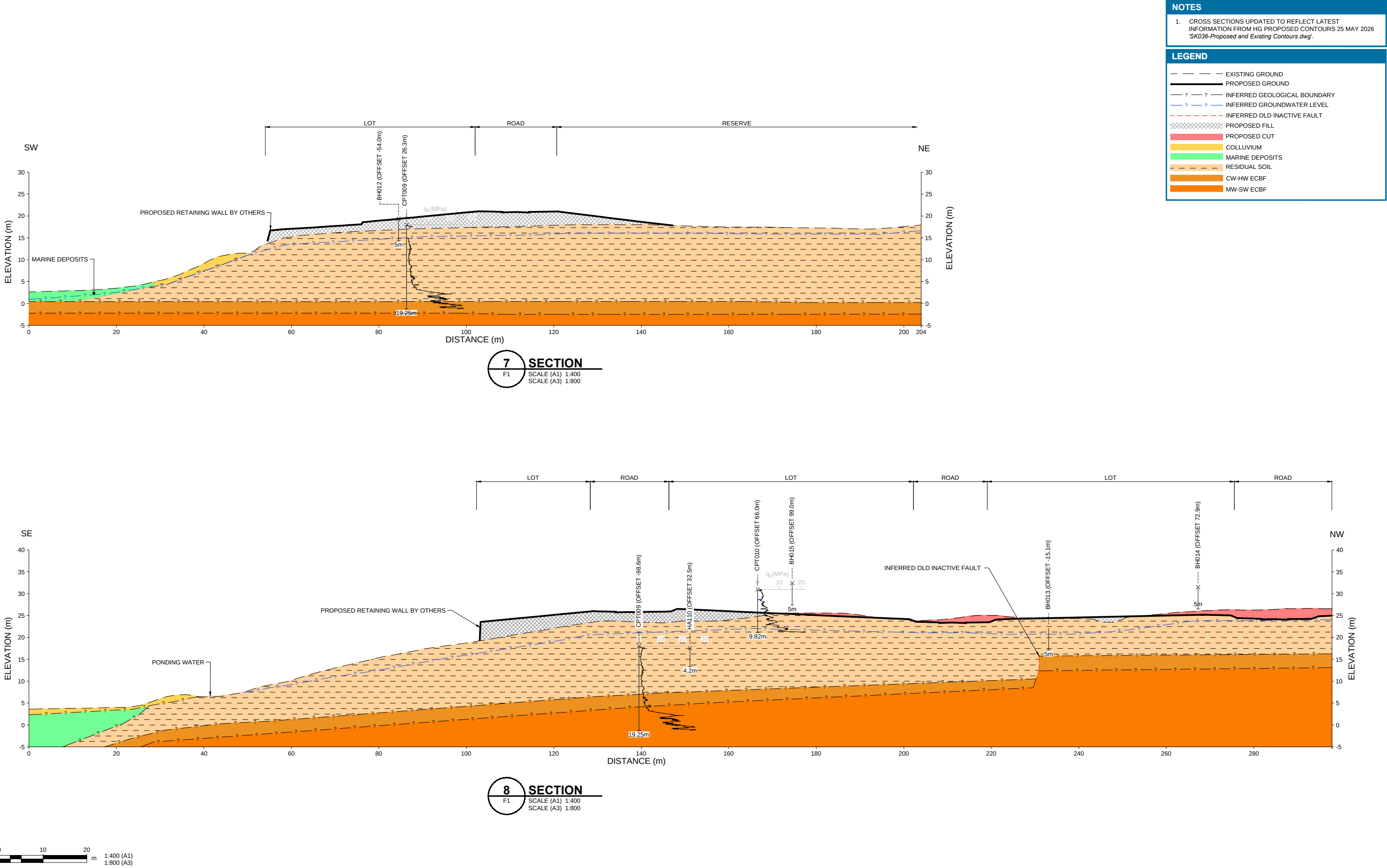


6 SECTION
F1 SCALE (A1) 1:400
SCALE (A3) 1:800



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PROJECT	BEACHLANDS SOUTH BALANCE LIVE-ZONE		
TITLE	GEOLOGICAL CROSS-SECTION 6		
SCALE (A1)	1:400	FIG No.	FIGURE 5
		REV	1



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PROJECT	BEACHLANDS SOUTH BALANCE LIVE-ZONE		
TITLE	GEOLOGICAL CROSS-SECTIONS 7 & 8		
SCALE (A1)	1:400	FIG No.	FIGURE 6
REV	1		

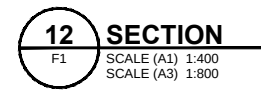
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- INFERRED GEOLOGICAL BOUNDARY
- INFERRED GROUNDWATER LEVEL
- PROPOSED FILL
- PROPOSED CUT
- COLLUVIUM
- RESIDUAL SOIL
- CW-HW ECBF
- MW-SW ECBF



1. CROSS SECTIONS UPDATED TO REFLECT LATEST INFORMATION FROM HG PROPOSED CONTOURS 25 MAY 2026
'SK036-Proposed and Existing Contours.dwg'.

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 ? ? ? INFERRED GROUNDWATER LEVEL
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 COLLUVIUM
 ALLUVIUM
 RESIDUAL SOIL
 CW-HW ECBF
 MW-SW ECBF

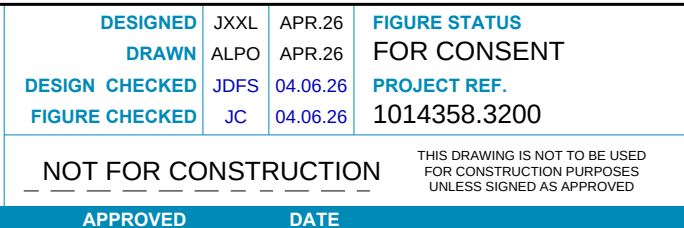


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 ————— PROPOSED GROUND
 — ? — ? — INFERRED GEOLOGICAL BOUNDARY
 — ? — ? — INFERRED GROUNDWATER LEVEL
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 — — — — — RESIDUAL SOIL
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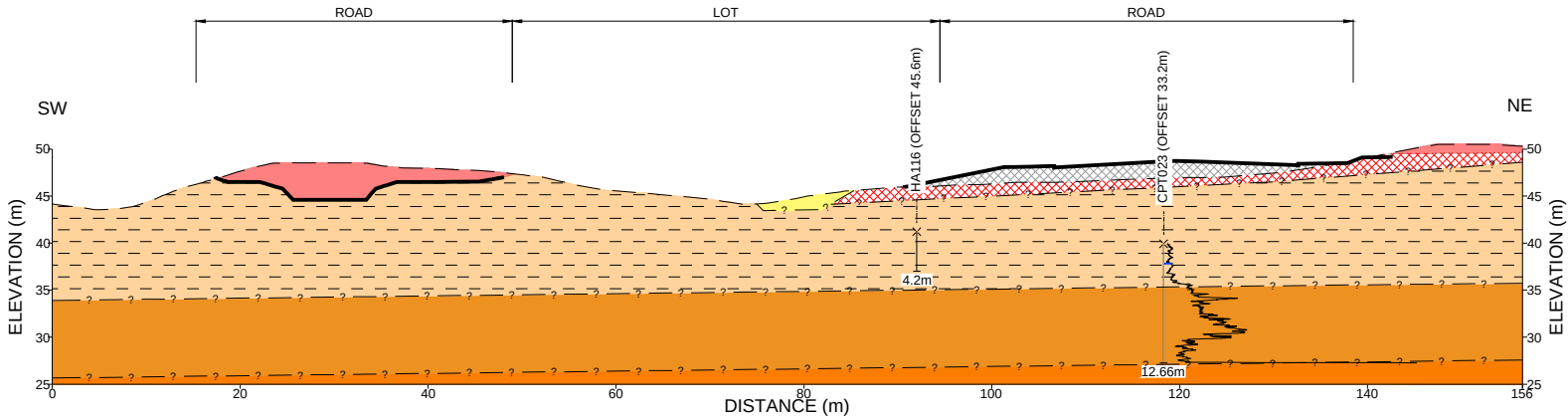


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PROJECT	BEACHLANDS SOUTH BALANCE LIVE-ZONE		
TITLE	GEOLOGICAL CROSS-SECTION 15		
SCALE (A1)	1:400	FIG No.	FIGURE 9
		REV	1

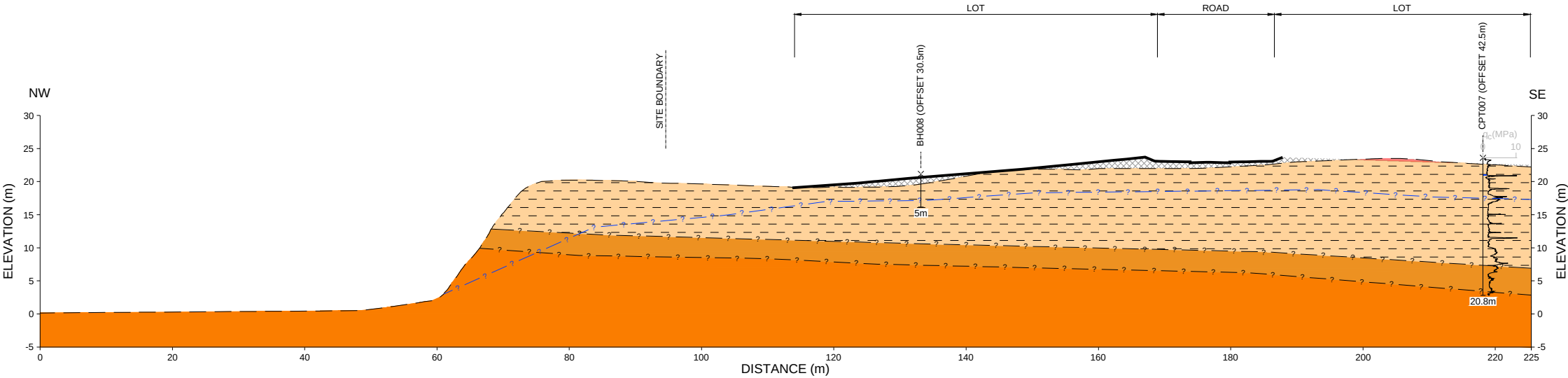
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1. CROSS SECTIONS UPDATED TO REFLECT LATEST INFORMATION FROM HG PROPOSED CONTOURS 25 MAY 2026 'SK036-Proposed and Existing Contours.dwg'.

- LEGEND**
- EXISTING GROYNDE
 - PROPOSED GROUND
 - INFERRED GEOLOGICAL BOUNDARY
 - INFERRED GROUNDWATER LEVEL
 - PROPOSED FILL
 - PROPOSED CUT
 - HISTORIC FILL
 - ALLUVIUM
 - RESIDUAL SOIL
 - CW-HW ECBF
 - MW-SW ECBF



17 SECTION
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SCALE (A1) 1:400
SCALE (A3) 1:800



18 SECTION
F1
SCALE (A1) 1:400
SCALE (A3) 1:800



1:400 (A1)
1:800 (A3)



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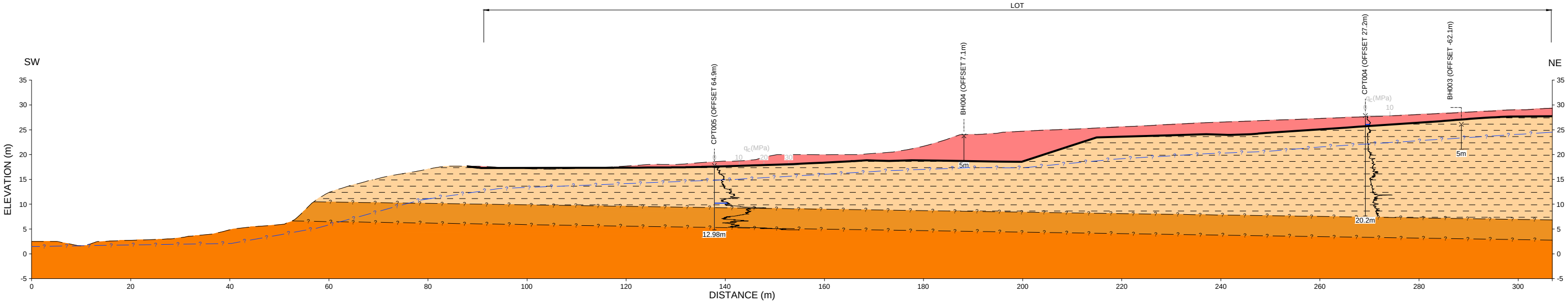
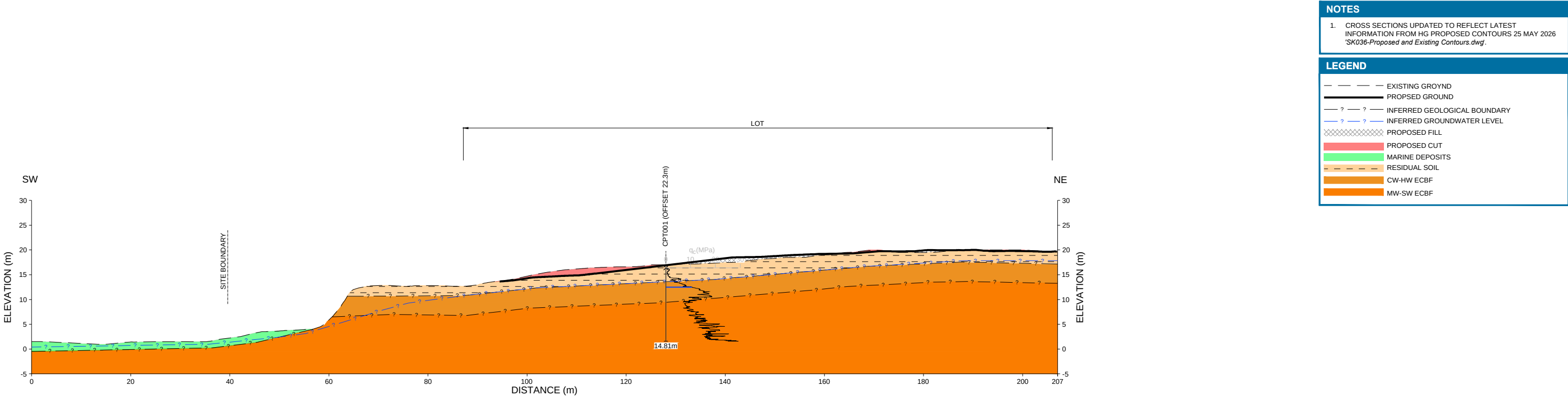
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PROJECT **BEACHLANDS SOUTH BALANCE LIVE-ZONE**

TITLE
GEOLOGICAL CROSS-SECTIONS 17 & 18

SCALE (A1) 1:400

FIG No. FIGURE 10

REV 1



1:400 (A1)

1:800 (A3)



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PROJECT **BEACHLANDS SOUTH BALANCE LIVE-ZONE**

TITLE

GEOLOGICAL CROSS-SECTIONS 19 & 20

SCALE (A1) 1:400

FIG No. FIGURE 11

REV 1

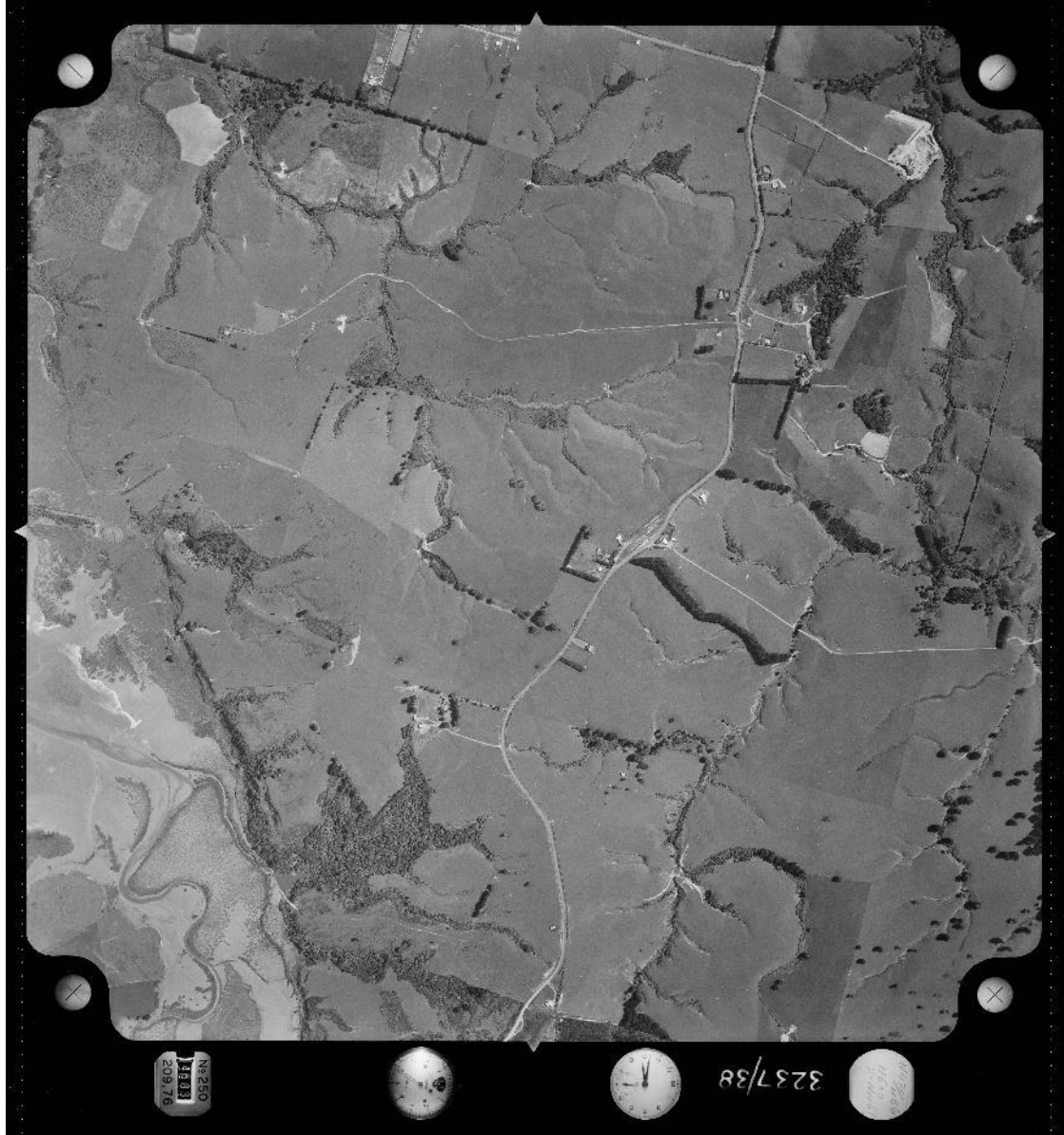


1939 Aerial Photograph



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1955 Aerial Photograph



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1961 Aerial Photo



1972 Aerial Photograph



Aerial Photograph 1980



1996 Aerial Photograph



2001 Satellite Imagery



2017 Satellite Imagery



2025 Aerial Photograph

Appendix B Historic Geotechnical Investigation Logs

Lander's 2016 Investigation Logs:

- **Lander_BH01 to Lander_BH12**

T+T's 2021 Investigation Logs:

- **TT_HA100 to TT_HA116**
- **CPT100 to CPT102**

For T+T's 2025 Stage 1 Consented Earthworks Area Investigation Logs, refer to:

- **T+T Stage 1 Geotechnical Report (Ref. [9])**
- **T+T Stage 1 Geotechnical Letter (Ref. [10])**

HAND AUGER LOG

HOLE Id: **HA108**

SHEET: 1 OF 1

PROJECT: Geotechnical Engineering		LOCATION: 110 Jack Lachlan Drive, Beachlands 2571		JOB No.: 1014358.3000	
CO-ORDINATES: 5914649 mN (NZTM2000) 1777242 mE		DRILL TYPE: 50mm Hand Auger		HOLE STARTED: 24/11/2021	
R.L.: 19m		METHOD: Hand auger		HOLE FINISHED: 24/11/2021	
DATUM: NZVD2016				DRILLED BY: T+T	
				LOGGED BY: CMCD	
				CHECKED: NBK	

GEOLOGICAL	METHOD OBSERVATIONS										ENGINEERING DESCRIPTION												
STRATIGRAPHY / ENG GEOLOGICAL UNIT / ADDITIONAL OBSERVATIONS	WATER	CORE RECOVERY (%)	METHOD	SCALA PENETROMETER (Blows/100mm)										TESTS	SAMPLES	RL (m)	DEPTH (m)	GRAPHIC LOG	WEATHERING CLASSIFICATION	MOISTURE CLASSIFICATION	CONSISTENCY / DENSITY CLASSIFICATION	ESTIMATED SOIL SHEAR STRENGTH (kPa)	DESCRIPTION
				0	1	2	3	4	5	6	7	8	9										
Fill																						0.00m: SILT, some clay, trace rootlets; brown. Stiff, dry, low plasticity.	
														● >207 kPa								0.25m: Clayey SILT; orange brown. Very stiff to hard, moist, medium plasticity.	
Residual East Coast Bays Formation Soils														● >207 kPa	18	1						0.80m: Silty CLAY; light grey streaked orange. Very stiff to hard, moist, high plasticity.	
														● 144/78 kPa								1.50 - 2.00m: pink streaks.	
														● 135/58 kPa	17	2						2.00m: SILT, some clay; brown mottled orange. Very stiff, wet, low to medium plasticity.	
														● 135/52 kPa								2.40 - 2.70m: pink streaked grey.	
														● 109/40 kPa	16	3						2.70m: Clayey SILT; pink streaked grey. Very stiff, wet, medium plasticity.	
														● 149/49 kPa								3.70 - 3.80m: orange.	
													● 189/61 kPa	15	4								
														14	5							4.2m: Target depth	

COMMENTS:

Hole Depth
4.2m

HAND AUGER PHOTOS

BOREHOLE No.: **HA108**

SHEET: 1 OF 1

PROJECT: Geotechnical Engineering		LOCATION: 110 Jack Lachlan Drive, Beachlands 2 JOB No.: 1014358.3000	
CO-ORDINATES: (NZTM2000)	5914649 mN 1777242 mE	DRILL TYPE: 50mm Hand Auger	HOLE STARTED: 24/11/2021
R.L.:	19m	METHOD: Hand auger	HOLE FINISHED: 24/11/2021
DATUM:	NZVD2016	LOGGED BY: CMCD	CHECKED: NBK



0.00-4.20m

HAND AUGER LOG

HOLE ID: HA109

SHEET: 1 OF 1

PROJECT: Geotechnical Engineering	LOCATION: 110 Jack Lachlan Drive, Beachlands 2571	JOB No.: 1014358.3000
CO-ORDINATES: 5914403 mN (NZTM2000) 1777234 mE	DRILL TYPE: 50mm Hand Auger	HOLE STARTED: 24/11/2021
R.L.: 23m	METHOD: Hand auger	HOLE FINISHED: 24/11/2021
DATUM: NZVD2016		DRILLED BY: T+T
		LOGGED BY: CMCD
		CHECKED: NBK

GEOLOGICAL		METHOD OBSERVATIONS										ENGINEERING DESCRIPTION											
STRATIGRAPHY / ENG GEOLOGICAL UNIT / ADDITIONAL OBSERVATIONS	WATER	CORE RECOVERY (%)	METHOD	SCALA PENETROMETER (Blows/100mm)										TESTS	SAMPLES	RL (m)	DEPTH (m)	GRAPHIC LOG	WEATHERING CLASSIFICATION	MOISTURE CLASSIFICATION	CONSISTENCY / DENSITY CLASSIFICATION	ESTIMATED SOIL SHEAR STRENGTH (Sk, kPa)	DESCRIPTION
				0	1	2	3	4	5	6	7	8	9										
Fill																		M	St			0.00m: Clayey SILT, some rootlets; brown. Stiff, moist, medium plasticity.	
																						W	VSt-H
Residual East Coast Bays Formation Soils	DRY 24/11/2021	100	HA															M	VSt			0.65m: Silty CLAY; orange streaked light grey. Very stiff, moist, high plasticity.	
																						VSt-H	1.50m: CLAY, some silt; grey streaked pink. Very stiff to hard, moist, high plasticity.
																							2.90m: Silty CLAY; light grey. Very stiff, wet, high plasticity.
																							3.50m: Clayey SILT; light grey streaked orange. Very stiff, wet, medium plasticity.
																							4.2m: Target depth

COMMENTS:
Hole Depth 4.2m

HAND AUGER PHOTOS

PROJECT: Geotechnical Engineering		LOCATION: 110 Jack Lachlan Drive, Beachlands 2 JOB No.: 1014358.3000	
CO-ORDINATES: (NZTM2000)	5914403 mN 1777234 mE	DRILL TYPE: 50mm Hand Auger	HOLE STARTED: 24/11/2021
R.L.:	23m	METHOD: Hand auger	HOLE FINISHED: 24/11/2021
DATUM:	NZVD2016	LOGGED BY: CMCD	CHECKED: NBK



0.00-4.20m

HAND AUGER LOG

HOLE ID: HA110

SHEET: 1 OF 1

PROJECT: Geotechnical Engineering	LOCATION: 110 Jack Lachlan Drive, Beachlands 2571	JOB No.: 1014358.3000
CO-ORDINATES: 5914246 mN (NZTM2000) 1777561 mE	DRILL TYPE: 50mm Hand Auger	HOLE STARTED: 24/11/2021
R.L.: 27m	METHOD: Hand auger	HOLE FINISHED: 24/11/2021
DATUM: NZVD2016		DRILLED BY: T+T
		LOGGED BY: CMCD
		CHECKED: NBK

GEOLOGICAL		METHOD OBSERVATIONS										ENGINEERING DESCRIPTION											
STRATIGRAPHY / ENG GEOLOGICAL UNIT / ADDITIONAL OBSERVATIONS	WATER	CORE RECOVERY (%)	METHOD	SCALA PENETROMETER (Blows/100mm)										TESTS	SAMPLES	RL (m)	DEPTH (m)	GRAPHIC LOG	WEATHERING CLASSIFICATION	MOISTURE CLASSIFICATION	CONSISTENCY / DENSITY CLASSIFICATION	ESTIMATED SOIL SHEAR STRENGTH (kN/m²)	DESCRIPTION
				0	1	2	3	4	5	6	7	8	9										
Topsoil																		M	St		0.00m: Clayey SILT, some rootlets; brown. Stiff, moist, medium plasticity.		
Residual East Coast Bays Formation Soils	24/11/2021													● 98/61 kPa	26	1		W	St		0.20m: Silty CLAY; light grey streaked orange. Stiff to very stiff, moist, high plasticity.		
												● 92/43 kPa											
												● 61/35 kPa										1.20m: Clayey SILT; grey streaked orange. Stiff, wet, medium plasticity.	
												● >207 kPa	25	2								1.60m: SILT, some clay; brown mixed grey. Stiff, wet, low plasticity.	
												● >207 kPa											
												● 161/43 kPa	24	3			H					2.10m: Clayey SILT; grey streaked orange. Hard, wet, medium plasticity.	
												● 132/52 kPa											
												● 109/46 kPa	23	4			VSt					3.40m: Clayey SILT; dark grey. Very stiff, wet, medium plasticity.	

0.0-4.2m

COMMENTS:

Hole Depth
4.2m

HAND AUGER PHOTOS

BOREHOLE No.: **HA110**

SHEET: 1 OF 1

PROJECT: Geotechnical Engineering		LOCATION: 110 Jack Lachlan Drive, Beachlands 2 JOB No.: 1014358.3000	
CO-ORDINATES: (NZTM2000)	5914246 mN 1777561 mE	DRILL TYPE: 50mm Hand Auger	HOLE STARTED: 24/11/2021
R.L.:	27m	METHOD: Hand auger	HOLE FINISHED: 24/11/2021
DATUM:	NZVD2016	LOGGED BY: CMCD	CHECKED: NBK



0.00-4.20m

HAND AUGER LOG

HOLE Id: **HA111**

SHEET: 1 OF 1

PROJECT: Geotechnical Engineering		LOCATION: 110 Jack Lachlan Drive, Beachlands 2571		JOB No.: 1014358.3000	
CO-ORDINATES: 5914063 mN (NZTM2000) 1778205 mE		DRILL TYPE: 50mm Hand Auger		HOLE STARTED: 24/11/2021	
R.L.: 53m		METHOD: Hand auger		HOLE FINISHED: 24/11/2021	
DATUM: NZVD2016				DRILLED BY: T+T	
				LOGGED BY: CMCD	
				CHECKED: NBK	

GEOLOGICAL	METHOD OBSERVATIONS										ENGINEERING DESCRIPTION													
STRATIGRAPHY / ENG GEOLOGICAL UNIT / ADDITIONAL OBSERVATIONS	WATER	CORE RECOVERY (%)	METHOD	SCALA PENETROMETER (Blows/100mm)										TESTS	SAMPLES	RL (m)	DEPTH (m)	GRAPHIC LOG	WEATHERING CLASSIFICATION	MOISTURE CLASSIFICATION	CONSISTENCY / DENSITY CLASSIFICATION	ESTIMATED SOIL SHEAR STRENGTH (kPa)	DESCRIPTION	
				0	1	2	3	4	5	6	7	8	9											
Topsoil																						0.00m: Clayey SILT, some rootlets; brown. Stiff, moist, low plasticity.		
Fill																						0.15m: Clayey SILT, trace sand; brown mottled orange. Very stiff, moist, medium plasticity. Sand, fine.		
Residual East Coast Bays Formation Soils														● 164/89 kPa									0.50m: Silty CLAY; light grey streaked orange. Very stiff, moist, high plasticity.	
														● 149/75 kPa	52	1							1.10m: SILT, some clay; brown mottled orange. Very stiff, wet, low plasticity.	
														● 201/72 kPa									1.25 - 1.50m: pink with grey streaks.	
														● >207 kPa	51	2							1.50m: Clayey SILT; pink. Very stiff to hard, wet, medium plasticity.	
														● 201/72 kPa									2.60 - 3.40m: mottled pink and orange.	
														● >207 kPa	50	3								3.70 - 4.20m: pink mottled grey.
														● 149/75 kPa										
														● 164/38 kPa	49	4								
															48	5							4.2m: Target depth	

COMMENTS:

Hole Depth
4.2m

HAND AUGER PHOTOS

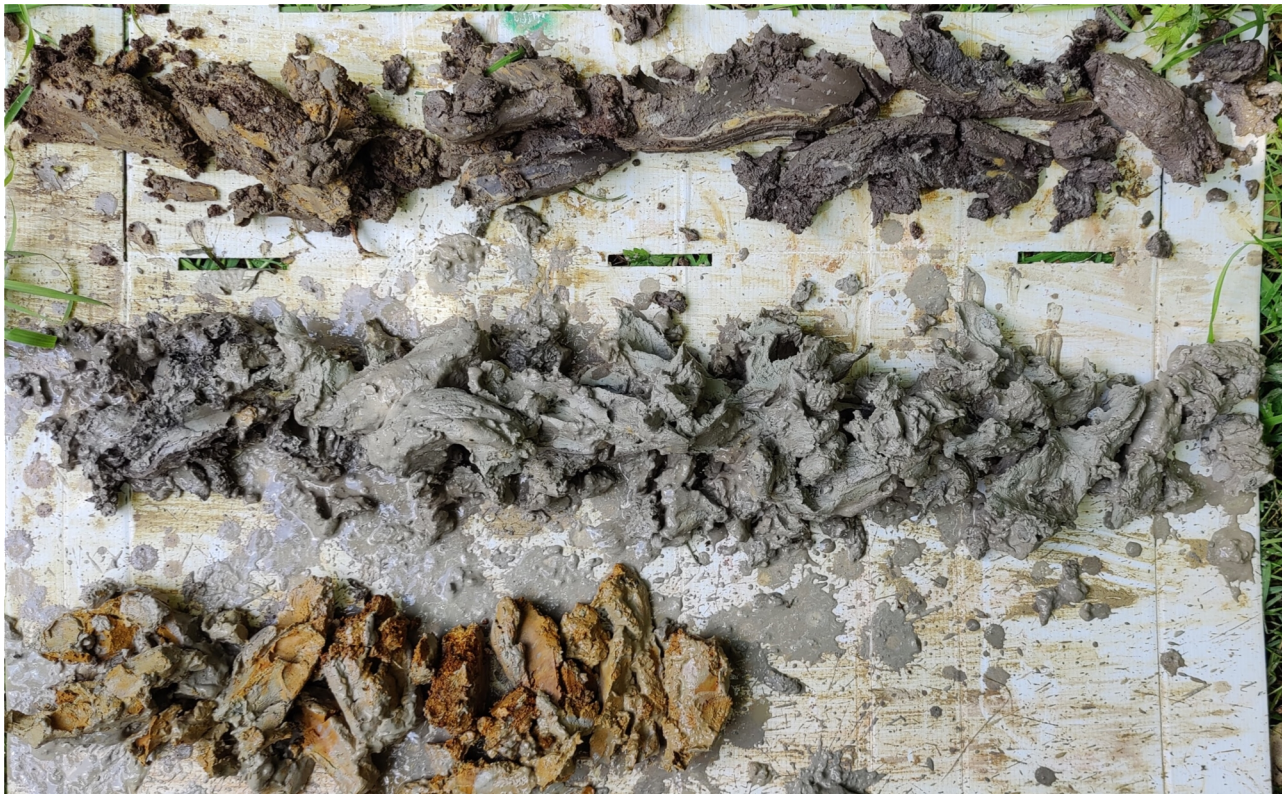
PROJECT: Geotechnical Engineering		LOCATION: 110 Jack Lachlan Drive, Beachlands 2 JOB No.: 1014358.3000	
CO-ORDINATES: (NZTM2000)	5914063 mN 1778205 mE	DRILL TYPE: 50mm Hand Auger	HOLE STARTED: 24/11/2021
R.L.:	53m	METHOD: Hand auger	HOLE FINISHED: 24/11/2021
DATUM:	NZVD2016	LOGGED BY: CMCD	CHECKED: NBK



0.00-4.20m

HAND AUGER PHOTOS

PROJECT: Geotechnical Engineering		LOCATION: 110 Jack Lachlan Drive, Beachlands 2 JOB No.: 1014358.3000	
CO-ORDINATES: (NZTM2000)	5914594 mN 1779098 mE	DRILL TYPE: 50mm Hand Auger	HOLE STARTED: 09/12/2021
R.L.:	37m	METHOD: Hand auger	HOLE FINISHED: 09/12/2021
DATUM:	NZVD2016	LOGGED BY: CMCD	CHECKED: NBK



0.00-2.50m

HAND AUGER PHOTOS

PROJECT: Geotechnical Engineering		LOCATION: 110 Jack Lachlan Drive, Beachlands 2 JOB No.: 1014358.3000	
CO-ORDINATES: (NZTM2000)	5914313 mN 1779222 mE	DRILL TYPE: 50mm Hand Auger	HOLE STARTED: 09/12/2021
R.L.:	52m	METHOD: Hand auger	HOLE FINISHED: 09/12/2021
DATUM:	NZVD2016	LOGGED BY: CMCD	CHECKED: NBK



0.00-4.20m



Legend and/or Notes:

Hand Auger
Borehole

BASE PLAN SOURCE: AUCKLAND COUNCIL GIS DATABASE, DATED 26/01/16

revision	description				drawn	approved	date	0120240360480 Horizontal Scale (metres) 0120240360480 Vertical Scale (metres)	drawn	TT	LANDER geotechnical	client:	110 FORMOSA (NZ) LIMITED	
					approved	SGL	project:		FORMOSA, JACK LACHLAN DRIVE, BEACHLANDS					
					date	02/02/16	title:		SITE PLAN (without contours)					
					scale	1:8000	project no:		J 00253	figure no:		01		
					original size	A3								

template revision: 1/2000 (1/01/2014)

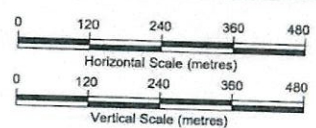


Legend and/or Notes:

Hand Auger Borehole

BASE PLAN SOURCE: AUCKLAND COUNCIL GIS DATABASE, DATED 26/01/16

revision	description	drawn	approved	date



drawn	TT
approved	SGL
date	02/02/16
scale	1:8000
original size	A3



client:	110 FORMOSA (NZ) LIMITED
project:	FORMOSA, JACK LACHLAN DRIVE, BEACHLANDS
title:	SITE PLAN (with contours)
project no:	J 00253
figure no:	02

Template revision: 1.0000 (10/12/14)

Client : 110 FORMOSA (NZ) LIMITED
 Project Location : FORMOSA, JACK LACHLAN DRIVE,
 BEACHLANDS
 Job Number: J00253

Auger Borehole No. BH1
 Sheet 1 of 12

Vane Head: 1900
 Logged By: TT
 Processor: TT
 Date: 21.01.16

Borehole Location: mN mE Ground R.L.
 Description: Refer to site plan

SOIL DESCRIPTION

TOPSOIL

silty CLAY, red, grey and orange mottled. Hard, moist, medium plasticity [FILL]
 becoming dark brown

silty CLAY, red and orange streaked grey. Hard, moist, medium plasticity [NATURAL]
 becoming brown/orange streaked grey, with some limonite silt inclusions

becoming red and orange streaked grey
 becoming slightly silty CLAY, very stiff, insensitive

becoming orange mottled light grey, hard

becoming light grey mottled orange

becoming very stiff

CLAY, grey. Hard, moist, high plasticity

clayey SILT, red mottled grey. Hard, moist, low plasticity

EOB at 4.0m. Target Depth.

Legend	Depth (m)	Standing Water Level	Vane Shear (kPa) peak / residual	Soil Sensitivity	Sample and Laboratory / Other Test Details
	0.5		208+		
	1.0		208+		
	1.5		208+		
	2.0		190/98	1.9	
	2.5		208+		
	3.0		157/92	1.7	
	3.5		208+		
	4.0		UTP		
	4.5				
	5.0				
	5.5				
	6.0				



Comments:
 Groundwater not encountered.
 UTP = unable to penetrate.
 EOB = end of borehole.

Borehole Diameter:
 50mm

Checked:
 LIB

Topsoil	Sand	Sandstone	Plutonic
Fill	Gravel	Siltstone	No Core
Clay	Organic	Limestone	
Silt	Pumice	Volcanic	

Client : 110 FORMOSA (NZ) LIMITED
 Project Location : FORMOSA, JACK LACHLAN DRIVE,
 BEACHLANDS
 Job Number: J00253

Auger Borehole No. BH2
 Sheet 2 of 12

Vane Head: 307
 Logged By: AB
 Processor: TT
 Date: 21.01.16

Borehole Location: mN mE Ground R.L.
 Description: Refer to site plan

SOIL DESCRIPTION

TOPSOIL

silty CLAY, brown. Very stiff, moist, high plasticity, moderately sensitive, with minor orange limonite silt inclusions and staining, with minor topsoil leaching [NATURAL]
 becoming grey mottled orange/brown

becoming insensitive
 becoming slightly silty CLAY, orange/brown mottled grey, with occasional limonite silt inclusions and staining

with occasional pink streaking

becoming medium plasticity, with some fine sand inclusions

becoming moderately sensitive
 becoming high plasticity, without fine sand inclusions

becoming medium plasticity, insensitive

becoming orange and pink streaked grey, high plasticity

EOB at 4.0m. Target Depth.

Legend	Depth (m)	Standing Water Level	Vane Shear (kPa) peak / residual	Soil Sensitivity	Sample and Laboratory / Other Test Details
	0.5		160/80	2.0	
	1.0		151/82	1.8	
	1.5		109/64	1.7	
	2.0		111/89	1.2	
	2.5		148/67	2.2	
	3.0		162/86	1.9	
	3.5		119/65	1.8	
	4.0		163/87	1.9	
	4.5				
	5.0				
	5.5				
	6.0				



Comments:
 Groundwater not encountered.
 UTP = unable to penetrate.
 EOB = end of borehole.

Borehole Diameter:	Topsoil	Sand	Sandstone	Plutonic
50mm	Fill	Gravel	Siltstone	No Core
Checked:	Clay	Organic	Limestone	
<i>Lib</i>	Silt	Pumice	Volcanic	

Client : 110 FORMOSA (NZ) LIMITED
 Project Location : FORMOSA, JACK LACHLAN DRIVE,
 BEACHLANDS
 Job Number: J00253

Auger Borehole No. BH3

Sheet 3 of 12

Vane Head: 1750
 Logged By: GB
 Processor: TT
 Date: 21.01.16

Borehole Location: mN mE Ground R.L.

Description: Refer to site plan

SOIL DESCRIPTION

TOPSOIL

silty CLAY, grey mottled orange/brown. Very stiff, moist, high plasticity, moderately sensitive [NATURAL]

becoming orange/brown mottled grey

becoming medium plasticity

clayey SILT, orange/brown mottled grey. Very stiff, moist, low to medium plasticity, insensitive

becoming moderately sensitive

becoming grey, low plasticity

becoming light brown/grey

becoming light brown/orange

becoming insensitive
 becoming light grey

at 4.0m, becoming moderately sensitive

EOB at 4.0m. Target Depth.

Legend
 Depth (m)
 Standing Water Level
 Vane Shear (kPa) peak/residual
 Soil Sensitivity
 Sample and Laboratory / Other Test Details

0.5

183/89

2.1

1.0

186+

1.5

175/90

1.9

2.0

151/77

2.0

2.5

159/73

2.2

3.0

178/89

2.0

3.5

162/106

1.5

4.0

122/60

2.0

4.5

5.0

5.5

6.0



Comments:
 Groundwater not encountered.
 UTP = unable to penetrate.
 EOB = end of borehole.

Borehole Diameter:

50mm

Checked:

GB

Topsoil

Fill

Clay

Silt

Sand

Gravel

Organic

Pumice

Sandstone

Siltstone

Limestone

Volcanic

Plutonic

No Core

Client : 110 FORMOSA (NZ) LIMITED Project Location : FORMOSA, JACK LACHLAN DRIVE, BEACHLANDS Job Number: J00253				Auger Borehole No. BH4 Sheet 4 of 12								
Vane Head: 1900		Logged By: TT		Processor : TT		Date: 21.01.16						
Borehole Location: mN mE Ground R.L.	Description: Refer to site plan											
SOIL DESCRIPTION				Legend	Depth (m)	Standing Water Level	Vane Shear (kPa) peak / residual	Soil Sensitivity	Sample and Laboratory / Other Test Details			
TOPSOIL												
slightly silty CLAY, orange streaked grey. Very stiff, moist, high plasticity, moderately sensitive, with trace limonite silt inclusions [FILL]					0.5		101/44	2.3				
becoming orange and light grey mottled yellow/brown												
becoming stiff					1.0		71/36	2.0				
becoming mottled dark brown, brown and orange												
with major gravel inclusions												
becoming silty CLAY, orange streaked light grey, with minor fine sand					1.5	▽	196/92	2.1				
becoming very stiff												
silty CLAY, orange streaked light grey. Hard, moist, high plasticity [NATURAL]					2.0		208+					
with some fine sand												
becoming orange					2.5		208+					
with occasional purple streaking					3.0		208+					
CLAY, grey. Very stiff, wet, high plasticity, insensitive					3.5		107/77	1.4				
at 4.0m, becoming stiff					4.0		89/47	1.9				
EOB at 4.0m. Target Depth.												
					4.5							
					5.0							
					5.5							
					6.0							
Comments: Groundwater encountered 1.4m UTP = unable to penetrate. EOB = end of borehole.				Borehole Diameter: 50mm	Topsoil		Sand		Sandstone		Plutonic	
				Checked: <i>LB</i>	Fill		Gravel		Siltstone		No Core	
					Clay		Organic		Limestone			
					Silt		Pumice		Volcanic			



Client : 110 FORMOSA (NZ) LIMITED
 Project Location : FORMOSA, JACK LACHLAN DRIVE,
 BEACHLANDS
 Job Number: J00253

Auger Borehole No. BH5
 Sheet 5 of 12

Vane Head: 307
 Logged By: AB
 Processor: TT
 Date: 21.01.16

Borehole Location: mN mE Ground R.L.
 Description: Refer to site plan

SOIL DESCRIPTION

TOPSOIL

clayey SILT with some fine sand, mottled brown and grey. Very stiff, moist, medium to low plasticity, sensitive, with some topsoil leaching [FILL]
 at 0.4m, without topsoil leaching
 becoming silty CLAY with minor fine sand, mottled brown, grey and orange, medium plasticity

becoming dark brown

becoming mottled dark brown, orange/brown and grey

becoming moderately sensitive

becoming grey streaked orange, with some limonite streaking and silt inclusions

becoming stiff
 becoming dark brown

silty CLAY, dark brown/grey. Very stiff, moist to wet, high plasticity, moderately sensitive, with minor rootlet inclusions [NATURAL]
 becoming slightly silty CLAY, orange streaked grey, moist, without rootlet inclusions

becoming grey

with minor dark orange limonite silt inclusions

EOB at 4.0m. Target Depth.

Legend	Depth (m)	Standing Water Level	Vane Shear (kPa) peak / residual	Soil Sensitivity	Sample and Laboratory / Other Test Details
	0.5		154/26	5.9	
	1.0		107/19	5.6	
	1.5		152/67	2.3	
	2.0		84/24	3.5	
	2.5		114/35	3.3	
	3.0		121/56	2.2	
	3.5		125/40	3.1	
	4.0		133/58	2.3	
	4.5				
	5.0				
	5.5				
	6.0				



Comments:
 Groundwater not encountered.
 UTP = unable to penetrate.
 EOB = end of borehole.

Borehole Diameter:

50mm

Checked: LB

Topsoil	Sand	Sandstone	Plutonic
Fill	Gravel	Siltstone	No Core
Clay	Organic	Limestone	
Silt	Pumice	Volcanic	

Client : 110 FORMOSA (NZ) LIMITED
 Project Location : FORMOSA, JACK LACHLAN DRIVE,
 BEACHLANDS
 Job Number: J00253

Auger Borehole No. BH6
 Sheet 6 of 12

Vane Head: 1750
 Logged By: GB
 Processor: TT
 Date: 21.01.16

Borehole Location: mN mE Ground R.L.
 Description: Refer to site plan

Legend	Depth (m)	Standing Water Level	Vane Shear (kPa) peak / residual	Soil Sensitivity	Sample and Laboratory / Other Test Details
SOIL DESCRIPTION					
TOPSOIL					
silty CLAY, grey/brown mottled orange/brown. Very stiff, moist, medium plasticity [NATURAL]					
becoming orange/brown mottled grey, high plasticity					
becoming moderately sensitive					
becoming light grey, with minor orange/brown streaking and mottles					
becoming insensitive					
becoming moderately sensitive					
becoming low to medium plasticity					
clayey SILT, orange/brown streaked light grey. Very stiff, moist, low to medium plasticity					
becoming low plasticity					
becoming light grey, moderately sensitive					
silty CLAY, light grey mottled orange/brown. Very stiff, wet, low to medium plasticity					
clayey SILT, orange streaked light grey. Very stiff, wet, low plasticity at 3.6m, becoming moist					
EOB at 4.0m. Target Depth.					



Comments:
 Groundwater not encountered.
 UTP = unable to penetrate.
 EOB = end of borehole.

Borehole Diameter:	50mm	Topsoil	Sand	Sandstone	Plutonic
Checked:	GB	Fill	Gravel	Siltstone	No Core
		Clay	Organic	Limestone	
		Silt	Pumice	Volcanic	

Client : 110 FORMOSA (NZ) LIMITED
 Project Location : FORMOSA, JACK LACHLAN DRIVE,
 BEACHLANDS
 Job Number: J00253

Auger Borehole No. BH7
 Sheet 7 of 12

Borehole Location: mN mE Ground R.L.
 Description: Refer to site plan

Vane Head: 1750
 Logged By: GB
 Processor: TT
 Date: 21.01.16

SOIL DESCRIPTION

TOPSOIL

silty CLAY, mottled orange/brown and light grey. Very stiff, moist, medium plasticity [NATURAL]

becoming orange/brown mottled grey, high plasticity

becoming insensitive

becoming moderately sensitive

clayey SILT, orange/brown mottled light grey. Very stiff, moist, low to no plasticity
 at 2.1m, becoming orange/brown, with occasional grey specking
 at 2.3m, becoming grey mottled orange/brown

becoming orange/brown mottled grey

becoming grey/blue

EOB at 4.0m. Target Depth.

Legend	Depth (m)	Standing Water Level	Vane Shear (kPa) peak / residual	Soil Sensitivity	Sample and Laboratory / Other Test Details
	0.5		186+		
	1.0		162/101	1.6	
	1.5		173/82	2.1	
	2.0		186+		
	2.5		186+		
	3.0		UTP		
	3.5		UTP		
	4.0		UTP		
	4.5				
	5.0				
	5.5				
	6.0				



Comments:
 Groundwater not encountered.
 UTP = unable to penetrate.
 EOB = end of borehole.

Borehole Diameter:	Topsoil	Sand	Sandstone	Plutonic
50mm	Fill	Gravel	Siltstone	No Core
Checked:	Clay	Organic	Limestone	
GB	Silt	Pumice	Volcanic	

Client : 110 FORMOSA (NZ) LIMITED
Project Location : FORMOSA, JACK LACHLAN DRIVE, BEACHLANDS
Job Number: J00253

Auger Borehole No. BH8
 Sheet 8 of 12

Borehole Location: mN mE Ground R.L.
Description: Refer to site plan

Vane Head: 307
Logged By: AB
Processor : TT
Date: 21.01.16

SOIL DESCRIPTION

TOPSOIL

silty CLAY, mottled brown, dark brown and grey. Very stiff, dry, medium plasticity, moderately sensitive, with rootlet inclusions, with minor red/pink streaking [FILL]

becoming dark brown, with some organic staining

becoming orange mottled brown, hard, moist, high plasticity becoming dark brown

becoming light grey and pink streaked orange/brown, with occasional limonite silt inclusions

becoming streaked light grey, orange and pink, very stiff

becoming dark brown, hard, medium plasticity, with some organic staining

silty CLAY, dark brown. Very stiff, moist, medium plasticity [NATURAL]

becoming grey mottled orange/brown

becoming mottled orange/brown and grey, insensitive

becoming orange/brown streaked light grey

with minor pink streaking

becoming pink streaked light grey

becoming streaked pink and light grey

becoming grey streaked pink

EOB at 4.0m. Target Depth.

Legend	Depth (m)	Standing Water Level	Vane Shear (kPa) peak / residual	Soil Sensitivity	Sample and Laboratory / Other Test Details
	0.5		163/54	3.0	
	1.0		229+		
	1.5		184/71	2.6	
	2.0		229+		
	2.5		179/100	1.8	
	3.0		166/93	1.8	
	3.5		175/100	1.8	
	4.0		175/100	1.8	
	4.5				
	5.0				
	5.5				
	6.0				



Comments:
 Groundwater not encountered.
 UTP = unable to penetrate.
 EOB = end of borehole.

Borehole Diameter:

50mm

Checked:

GB

Topsoil	Sand	Sandstone	Plutonic
Fill	Gravel	Siltstone	No Core
Clay	Organic	Limestone	
Silt	Pumice	Volcanic	

Client : 110 FORMOSA (NZ) LIMITED
 Project Location : FORMOSA, JACK LACHLAN DRIVE,
 BEACHLANDS
 Job Number: J00253

Auger Borehole No. BH9
 Sheet 9 of 12

Borehole Location: mN mE Ground R.L.

Description: Refer to site plan

Vane Head: 1900
 Logged By: TT
 Processor: TT
 Date: 21.01.16

SOIL DESCRIPTION

TOPSOIL

clayey SILT, orange mottled brown/grey. Hard, dry, low plasticity [NATURAL]

silty CLAY, orange mottled light grey. Hard, dry to moist, medium plasticity

becoming moist, high plasticity
 becoming very stiff, moderately sensitive

becoming hard
 with moderately thin orange limonite band

sandy SILT with trace clay, orange mottled grey/brown. Loose, moist, no plasticity

at 2.4m, becoming orange
 becoming grey

silty CLAY, dark grey. Hard, moist, medium plasticity [TRANSITION TO BEDROCK]

with minor fine sand

slightly clayey SAND, dark grey. Loose, moist, no plasticity

EOB at 4.0m. Target Depth.



Comments:
 Groundwater not encountered.
 UTP = unable to penetrate.
 EOB = end of borehole.

Borehole Diameter:

50mm

Checked:

GB

Topsoil

Fill

Clay

Silt

Sand

Gravel

Organic

Pumice

Sandstone

Siltstone

Limestone

Volcanic

Plutonic

No Core

Legend

Depth (m)

Standing Water Level

Vane Shear (kPa) peak / residual

Soil Sensitivity

Sample and Laboratory / Other Test Details

0.5

1.0

1.5

2.0

2.5

3.0

3.5

4.0

4.5

5.0

5.5

6.0

UTP

UTP

193/92

208+

UTP

UTP

208+

UTP

2.1

Client : 110 FORMOSA (NZ) LIMITED
 Project Location : FORMOSA, JACK LACHLAN DRIVE,
 BEACHLANDS
 Job Number: J00253

Auger Borehole No. BH10
 Sheet 10 of 12

Borehole Location: mN mE Ground R.L.
 Description: Refer to site plan

Vane Head: 307
 Logged By: AB
 Processor: TT
 Date: 21.01.16

SOIL DESCRIPTION		Legend	Depth (m)	Standing Water Level	Vane Shear (kPa) peak / residual	Soil Sensitivity	Sample and Laboratory / Other Test Details
TOPSOIL							
clayey SILT, blue/grey. Hard, moist, medium to low plasticity [NATURAL] at 0.3m, becoming slightly clayey SILT, light brown, low plasticity, with occasional fine gravel sized limonite inclusions at 0.5m, becoming moderately sensitive			0.5	▽	204/64	3.2	
silty CLAY, grey mottled orange/brown. Very stiff, moist, high plasticity, moderately sensitive, with minor limonite silt inclusions and staining			1.0		114/54	2.1	
with some coarse sand to fine gravel sized dark orange limonite silt clast inclusions			1.5		109/51	2.1	
becoming slightly silty CLAY, orange/brown streaked grey, wet, without limonite silt inclusions			2.0		121/48	2.1	
becoming grey mottled orange			2.5		125/58	2.2	
becoming silty CLAY, grey			3.0		229+		
SILT with trace sand, dark grey. Loose, moist, no plasticity			3.5		UTP		
clayey SILT, dark grey. Hard, moist, low to medium plasticity			4.0		UTP		
EOB at 4.0m. Target Depth.			4.5				
			5.0				
			5.5				
			6.0				



Comments:
 Groundwater encountered 1.7m
 UTP = unable to penetrate.
 EOB = end of borehole.

Borehole Diameter:	Topsoil	Sand	Sandstone	Plutonic
50mm	Fill	Gravel	Siltstone	No Core
Checked: LB	Clay	Organic	Limestone	
	Silt	Pumice	Volcanic	

Client : 110 FORMOSA (NZ) LIMITED Project Location : FORMOSA, JACK LACHLAN DRIVE, BEACHLANDS Job Number: J00253				Auger Borehole No. BH11 Sheet 11 of 12									
Borehole Location: mN mE Ground R.L.		Vane Head: 1900		Logged By: TT		Processor : TT		Date: 21.01.16					
Description: Refer to site plan		Legend		Depth (m)		Standing Water Level		Vane Shear (kPa) peak / residual		Soil Sensitivity		Sample and Laboratory / Other Test Details	
SOIL DESCRIPTION													
TOPSOIL													
clayey SILT, light orange and white mottled yellow/brown. Very stiff, dry, low to medium plasticity [FILL]													
becoming silty CLAY, light grey streaked orange, moist, medium to high plasticity, moderately sensitive				0.5				151/42		3.6			
becoming brown and light grey mottled orange/brown													
becoming mottled dark brown, light grey and orange				1.0				160/47		3.4			
becoming hard with minor topsoil streaking				1.5				208+					
becoming orange streaked grey, high plasticity													
becoming mottled dark brown, orange and grey, with trace fine to medium gravel inclusions				2.0				208+					
becoming slightly silty CLAY, dark green/brown, very stiff, moderately sensitive, with minor fine sand sized clast inclusions				2.5				104/44		2.4			
becoming dark brown													
silty CLAY, green/brown. Hard, moist, high plasticity, with trace organic staining [NATURAL]				3.0				208+					
becoming orange streaked grey													
becoming very stiff, insensitive				3.5				157/92		1.7			
becoming slightly silty CLAY at 4.0m, becoming sensitive													
EOB at 4.0m. Target Depth.				4.0				160/37		4.3			
				4.5									
				5.0									
				5.5									
				6.0									
Comments: Groundwater not encountered. UTP = unable to penetrate. EOB = end of borehole.				Borehole Diameter: 50mm		Topsoil		Sand		Sandstone		Plutonic	
						Fill		Gravel		Siltstone		No Core	
				Checked: GB		Clay		Organic		Limestone			
						Silt		Pumice		Volcanic			



Client : 110 FORMOSA (NZ) LIMITED
 Project Location : FORMOSA, JACK LACHLAN DRIVE,
 BEACHLANDS
 Job Number: J00253

Auger Borehole No. BH12
 Sheet 12 of 12

Borehole Location: mN mE Ground R.L.
 Description: Refer to site plan

Vane Head: 1750
 Logged By: GB
 Processor: TT
 Date: 21.01.16

SOIL DESCRIPTION

TOPSOIL

silty CLAY, orange mottled light brown/yellow. Very stiff, moist, high plasticity [NATURAL]

becoming grey and orange/brown mottled light brown, with some well cemented angular medium gravel sized inclusions

becoming grey streaked orange/brown, without gravel inclusions

with minor limonite silt inclusions
 with minor well cemented gravel inclusions
 becoming grey mottled light yellow/brown
 without gravel inclusions
 becoming sensitive

becoming grey/white specked orange/brown, with coarse sand sized silt clasts

becoming moderately sensitive, with well cemented medium to coarse gravel sized inclusions
 becoming grey mottled orange/brown, without gravel inclusions

becoming insensitive
 becoming slightly silty CLAY, orange/brown, with minor manganese oxide inclusions

CLAY, grey/white. Very stiff, moist, high plasticity

EOB at 4.0m. Target Depth.

Legend	Depth (m)	Standing Water Level	Vane Shear (kPa) peak / residual	Soil Sensitivity	Sample and Laboratory / Other Test Details
	0.5		186+		
	1.0		186+		
	1.5		186+		
	2.0		173/40	4.3	
	2.5		159/61	2.6	
	3.0		118/66	1.8	
	3.5		135/104	1.3	
	4.0				
	4.5				
	5.0				
	5.5				
	6.0				



Comments:
 Groundwater not encountered.
 UTP = unable to penetrate.
 EOB = end of borehole.

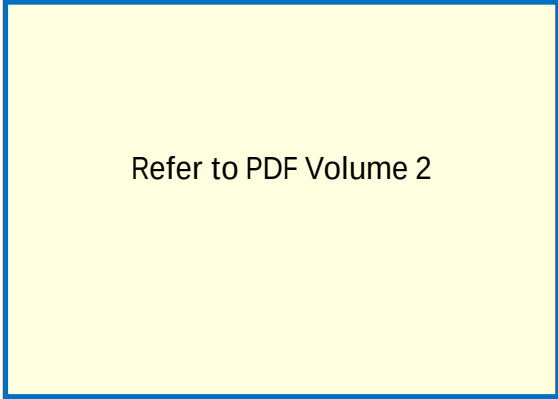
Borehole Diameter:
 50mm

Checked:
 GB

Topsoil	Sand	Sandstone	Plutonic
Fill	Gravel	Siltstone	No Core
Clay	Organic	Limestone	
Silt	Pumice	Volcanic	

Appendix C T+T 2026 Geotechnical Investigation Data

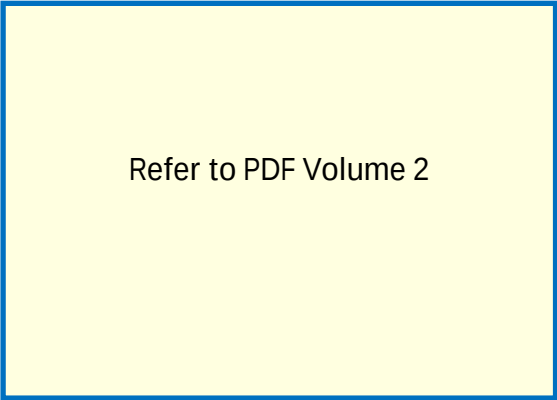
- **CPT Logs [CPT CPT-001 to CPT-023]**
- **HA Logs [BH001 to BH035]**
- **Piezometer Installation Records [BH001 to BH003, BH005 to BH009, BH011, BH015, BH017 to BH018, BH020 to BH022, BH025 to BH026, BH028, BH030, BH032, BH034]**
- **Groundwater Monitoring Graphs**
- **Geotechnics' Laboratory Testing Results [BH003, BH011, BH018, BH027, BH034]**



Refer to PDF Volume 2

Appendix D Liquefaction Analysis Outputs

- SLS
- ULS



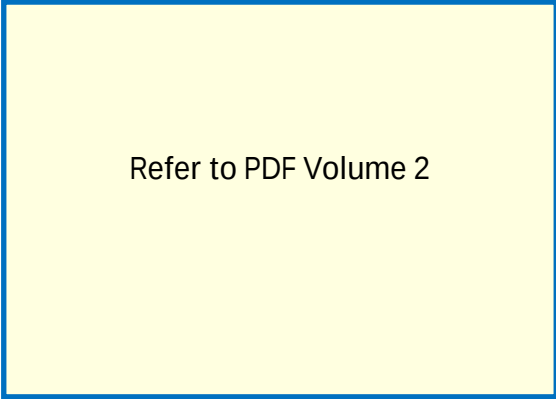
Refer to PDF Volume 2

Appendix E Slope Stability Analysis Outputs

Refer to 1014358.3200–Figure 1 Site Investigation Plan in Appendix A for slope stability cross-section locations:

- XS-1 (Western Cliff)
- XS-2
- XS-3
- XS-6 (Southern Cliff, relict scallop-shaped landslide feature)
- XS-7 (Southern Cliff)
- XS-8 (Southern Cliff)
- XS-9 (Southern Cliff)
- XS-11
- XS-12
- XS-15
- XS-18 (Western Cliff)
- XS-19 (Western Cliff)
- XS-20 (Western Cliff)

Refer to 1014358.3200–Figure 12 for indicative location and extent of the palisade wall, including a typical detail



Refer to PDF Volume 2

Appendix F Settlement Analysis Output

Refer to PDF Volume 2

Appendix G Draft Geotechnical Works Specification

Refer to PDF Volume 2

Appendix H HG Proposed Plans

HG's Drawings (DRAFT FOR INFORMATION ONLY):

- | | |
|---|---|
| • A2001228.00-HG-S1-SK-C-036 & -036A to -036C | PROPOSED AND EXISTING
CONTOURS [Rev A, 05.2026] |
| • A2515591.00-HG-S1-SK-C-037 | CUT/FILL VOLUME
[Rev A, 25.05.2026] |
| • A2515591.00-HG-S1-SK-C-078 & -078A to -078J | RETAINING WALL PLAN
[Rev A, 25.05.2026] |
| • A2515591.00-2700 to -2707 | RETAINING WALL LONGSECTION
[Rev A, 25.05.2026] |
| • A2515591.00-HG-S-1-SK-C-076 & -076A to -076F | COASTAL AREA STORMWATER
PLAN [Rev A, 25.05.2026] |

Refer to PDF Volumes 3 to 5

Appendix I Subsoil Drainage Typical Details

Refer to PDF Volume 2

Appendix J Cross-Sections at Proposed Basements Locations

Refer to PDF Volume 2

Appendix K Safety in Design Risk Assessment

Refer to PDF Volume 2

