



GEOTECHNICAL INVESTIGATION REPORT

Milldale Stages 14 - 17, Wainui East

Fulton Hogan Land Development Ltd
AK26-027AB Rev.1

24 July 2026

STATEMENTS OF QUALIFICATIONS AND EXPERIENCE

Chris Ritchie

I am a Principal at Gibb Ritchie Geotechnical Engineers Limited (GRGE). GRGE is a specialist geotechnical engineering consultancy based in Takapuna, Auckland, for which I am a founding director since 2025. I hold the qualifications of MSc (Engineering Geology) from The University of Auckland, which I completed in 2010. I am a Chartered Professional Engineer and Chartered Member of Engineering New Zealand. I have 17 years of professional experience in engineering geology and geotechnical engineering in the Auckland region. My experience has been primarily in land and building development and linear infrastructure, the last 14 years has been focused in the North Auckland area. Large portions of my work in this time have been focused on development of land and the investigation and assessment of roading corridors in Northland Allochthon terrain.

I have been engaged by Fulton Hogan Land Development Limited (FHLD) (the Applicant) to undertake an assessment of the Geotechnical Engineering associated with the Milldale Stages 14-17 project and to prepare this report in support of the substantive application under the Fast-track Approvals Act 2024 (FTAA).

While this is not an Environment Court proceeding, I confirm that I have prepared this assessment in accordance with the applicable professional and ethical obligations of my profession and that the opinions expressed are within my area of expertise.

Sam Gibb

I am a Principal Geotechnical Engineer at Gibb Ritchie Geotechnical Engineers Limited (GRGE). GRGE is a specialist geotechnical engineering consultancy based in Takapuna, Auckland, for which I am a founding director since 2025. I hold the qualification of Bachelor of Science in Civil Engineering from The University of Plymouth (UK), which I completed in 2008. I am a Chartered Professional Engineer and Chartered Member of Engineering New Zealand. I have 15 years' experience working as a Geotechnical Engineer in New Zealand, predominantly in the Auckland region. My main area of expertise is land development of various scales in Auckland. I have been involved in numerous site investigations, hazard analyses, remedial design, review, and construction monitoring of land development projects.

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LIMITATIONS AND USE OF THIS REPORT

This report has been prepared by Gibb Ritchie Geotechnical Engineers for the exclusive use of Fulton Hogan Land Development Ltd in relation to the Milldale Stages 14 - 17, Wainui East project. It has been prepared in accordance with the scope of work and the specific purpose agreed with the client.

The report should not be used for any other project, site, or purpose without the prior written consent of Gibb Ritchie Geotechnical Engineers. Where a party other than the named client seeks to rely on or use this report, they must first obtain such consent. Gibb Ritchie Geotechnical Engineers accepts no responsibility or liability to any other party who may use or rely on the contents of this report without authorisation.

This report is based on information available at the time of preparation, including site conditions observed and data obtained during the investigation. Subsurface conditions can vary significantly between investigation locations, and no warranty is given that the conditions described are representative of those across the entire site.

If you have any questions about the content or use of this report, please contact Gibb Ritchie Geotechnical Engineers for clarification.

Date	Revision	Comments
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2 June 2026	A	Internal Draft
5 June 2026	0	Draft for review
24 July 2026	1	Final

Name	Position	Signature
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Prepared by

Chris Ritchie	Principal Geotechnical Engineer (CPEng)
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Reviewed by

Sam Gibb	Principal Geotechnical Engineer (CPEng)
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1 INTRODUCTION

Gibb Ritchie Geotechnical Engineers Limited (GRGE) was engaged by Fulton Hogan Land Development Ltd to carry out a geotechnical investigation of several sites located between Cemetery Road and Endsley Rise, known as Milldale Stages 14 - 17, Wainui East. The Project relates to approximately 231ha of land centred around Wainui Road in Wainui, Auckland. The proposal provides for a comprehensively planned urban development including residential neighbourhoods, a supporting commercial node, transport and servicing infrastructure, open space, wetland restoration, an ecological restoration area and associated enabling works.

The scope of work and associated terms and conditions of our engagement were detailed in our services proposal referenced AK26-027AA Rev 0 dated 11 February 2026.

The application seeks approvals necessary to facilitate the staged development of the Project, including a suite of resource consents required under the FTAA framework.

The substantive application is supported by a comprehensive set of technical assessments and plans addressing the environmental, servicing and infrastructure matters associated with the proposal. This report should be read alongside the Planning Report and the appended technical reports and drawings lodged in support of the substantive application.

This report addresses matters relating to Geotechnical Engineering associated with the Project proposal for the land proposed for residential development only (south of the Orewa Awa). Refer to GRGE reports on the Water Infrastructure Platform (ref. AK26-027AC) and Reservoir (ref. AK26-027AD) for matters relating to Geotechnical Engineering associated with the Project proposal for the land proposed for infrastructure (north of the Orewa Awa). It seeks to describe the investigation completed, the ground conditions encountered, identify and quantify geotechnical risks to the development and to provide recommendations with respect to geotechnical aspects of the proposed land development including site preparation, temporary works, foundation construction and site / subsoil classification, as detailed in our proposal.

This report has been prepared having regard to the relevant statutory and technical framework applicable to the proposal, including the Auckland Code of Practice for Land Development and Subdivision: Chapter 2 – Earthworks and Geotechnical (Version 2) and Land development and subdivision infrastructure (NZS 4404:2010).

This report provides a level of investigation and preliminary analysis to broadly characterise the key geohazards pertinent to the development. We expect that future geotechnical investigation, analysis and design will be required prior to construction of each stage of the project.

2 SITE DETAILS

The application site encompasses approximately 231 hectares of land surrounding the existing Milldale development which is currently under construction. The application can be broken down into three broad areas as follows:

- Approximately 18ha of land north of Lysnar Road / Endsley Rise in the eastern portion of the site;
- Approximately 82ha of land south of the Orewa Awa and north Wainui Road and / or Milldale Stages 11 - 12; and
- Approximately 131ha of land north of the Orewa Awa (not covered by this report)

For the purpose of this report, we have divided the Project into three areas as shown on Figure 1:

- Cemetery Road Block (Stage 15), land holdings between existing Milldale Stage 12, Wainui Road and Cemetery Road (labelled 1 on Figure 1)
- Wainui Road North Block (Stage 14), parcels north of Wainui Road and south of the Orewa River (labelled 2 on Figure 1)
- Lysnar / Endsley Rise Block, parcels south of Wainui Road and east of Lysnar Road (Stage 16), and west of Sidwell Road (Stage 17), labelled 3 on Figure 1.

3 PROPOSED DEVELOPMENT

The Milldale Stages 14-17 project seeks approvals for an urban development on the site. The proposal provides for up to 1,425 residential dwellings across a range of housing typologies and densities. In addition, the proposal includes provision for a small commercial area, transport infrastructure, servicing and stormwater infrastructure, subdivision, earthworks, stream works, and neighbourhood parks. To the north of the Orewa Awa, the project includes a large-scale ecological restoration area, including walking and cycling trails to enhance recreation and amenity.

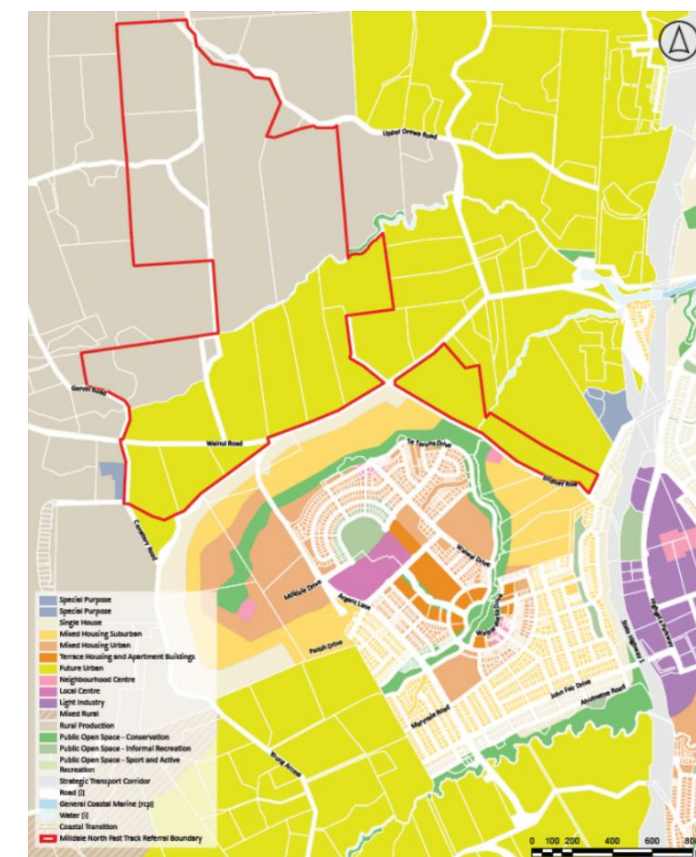
Development of the Project will require a suite of approvals under the FTAA, including resource consents associated with subdivision, land use activities, bulk earthworks, infrastructure works, stream works, water take, diversion and discharge; along with NES for contamination and freshwater.

A full description of the proposal is provided in the Planning Report and technical reports and drawings appended to the substantive application and should be read in conjunction with this report.

Figure 1 Site Plan



Figure 2 Development Plan



4 DESKTOP STUDY

4.1 Related Documents

The following documents were reviewed during preparation of this report:

- Geotechnical Investigation for Milldale Stage 8, prepared by CMW Geosciences referenced AKL2022-0029AE Rev 3, dated 4 September 2024.
- Geotechnical Investigation for Milldale Stages 10-13, prepared by CMW Geosciences referenced AKL2024-0257AB Rev 3, dated 25 March 2025.
- Geotechnical Investigation for Milldale North, prepared by CMW Geosciences referenced AKL2024-0257AB Rev 3, dated 25 March 2025.
- Geotechnical Investigation for Milldale Temporary Wastewater Treatment Facility, prepared by CMW Geosciences referenced AK2024-0185AC Rev 1 dated 26 February 2025.

4.2 Existing Information

The following geotechnical intrusive investigation exists within the footprint of the proposed development.

Table 1 Existing intrusive geotechnical investigation

Test IDs	Test Type	No. of Tests
MH07-18	Machine Boreholes	1
CMW CPT01 to 12-22	Cone Penetration Tests	12
CMW HA01 to 10-22, HA01 to 05-24	Hand Auger Boreholes	15

4.3 Site History

In order to determine the site history, historical area photographs from the website Retrolens and Auckland Council GIS were reviewed between 1940 and present day, key features are summarised below for each area:

Cemetery Road Block (Stage 15)

- 1940 the block is generally farmland with scattered trees
- 1950 the blocks are largely unchanged but extensive quarrying on the block to the south
- 1970 a dwelling is constructed in the northwestern corner (intersection of Wainui Road and Cemetery Road)
- 1980 some evidence of potential tipping as shown in Figure 3 near a watercourse
- 1990 development of the western portion of the block into horticultural use, including a large pond dug near the southern boundary and a second northern boundary along the boundary with Wainui Road
- 2024 removal of all buildings, and filling of large pond to the south.

Wainui Road North Block (Stage 14)

- 1940 the site is a large farm existing, farm stead near the Orewa Awa
- 1950 new ponds along southern boundary at head of gullies, new dwellings and wind break near the road along the southern boundary
- 1970 new dwelling near the western boundary and horticultural area near the northwestern boundary
- 1980 various areas of potential tipping noted
- 2024 demolition of the existing buildings near the northern boundary.

Endsley Rise Block (Stages 16, 17)

- 1940 the block on Lysnar Road is generally farmland with some scattered trees on the Endsley Rise block.
- 1950 both blocks are completely cleared of trees.
- 2000 trees are established on the Lysnar block and the stand of Poplars are present on the Endsley Rise block
- 2024 no changes from 2000 on the Endsley Rise block, construction laydown present on the Lysnar Road block.

Figure 3 Potential Uncertified Fill Area, Cemetery Road Block, 1970s

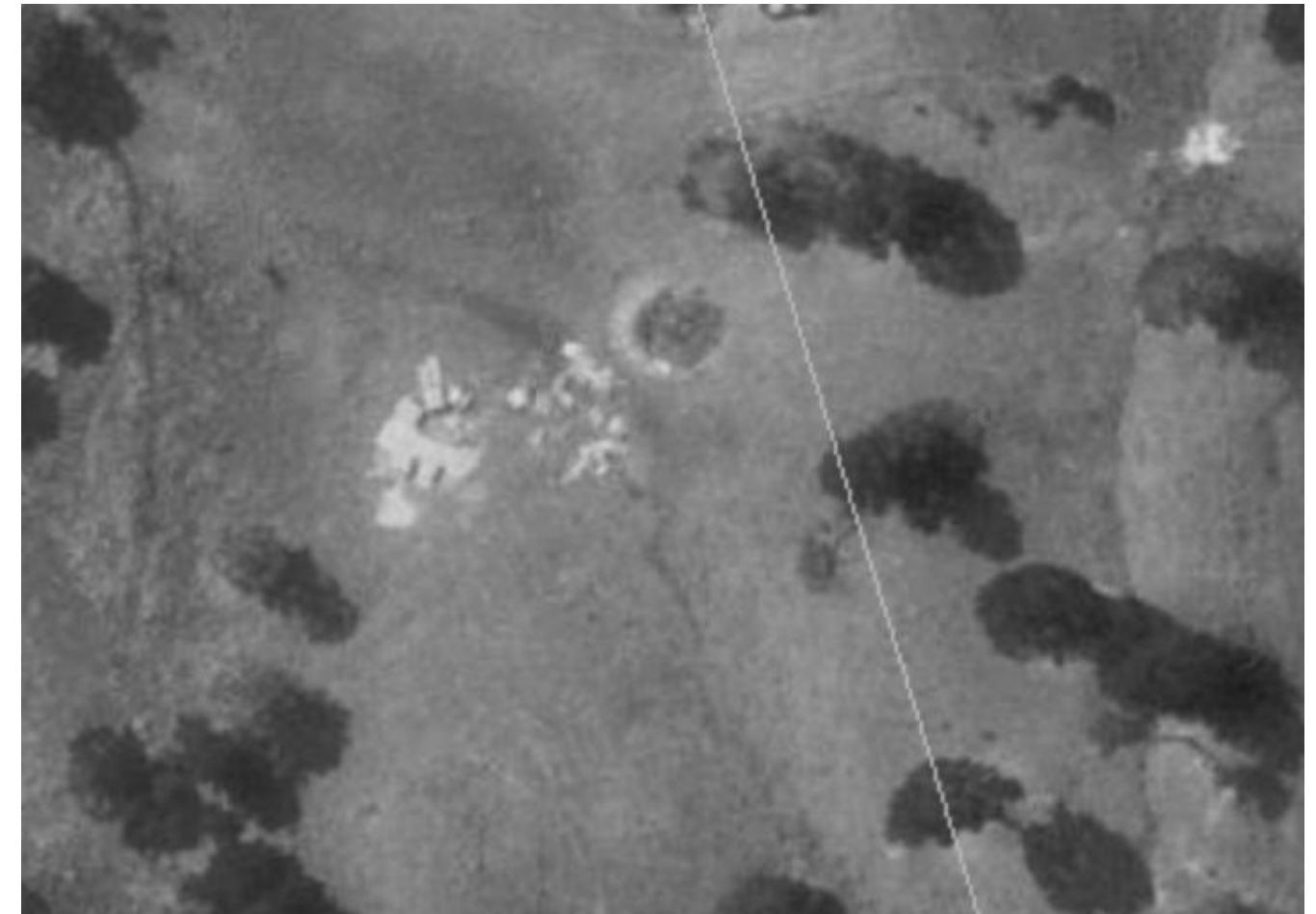


Figure 4 Potential Uncertified Fill Deposits, Wainui Road North, 1970s



5.1 Published Geology

Published geological maps¹ for the area depict the regional geology for the area as comprising either Middle Pleistocene - Late Pleistocene River and Hill Slope Deposits or Hukerenui Mudstone of the Mangakahia Complex of the Northland Allochthon. A description of these geological units can be found in the Tables below.

Table 2: Summary of Typical Conditions of Mapped Geological Units

Middle Pleistocene - Late Pleistocene River and Hill Slope Deposits	
QMap Code and description	IQa Predominantly pumiceous sand, silt, mud and clay, with interbedded gravel and peat.
Formation	River and beach deposits
Typical Composition, Weathering/ Layering and Variability	Interbedded clays, silts and sands with and without organics, dependant on the origin. Some horizons of pumiceous sands from distal eruptive centres. Can be highly variable.
Behaviour, Instability Mechanism(s) and Principal Potential Geohazards	Liquefaction (if sandy lenses are present), typically too old to liquefy. Lateral Spread if liquefiable near a free face. Prone to comparatively poor bearing capacity characteristics. Secondary creep settlements due to decomposing organic content. Typically, partially consolidated due to the age of the deposits.

Hukerenui Mudstone of the Mangakahia Complex of the Northland Allochthon	
QMap Code and description	KKh - Red, brown, green and grey, typically noncalcareous, commonly highly sheared mudstone, with small serpentinite bodies.
Formation	Moderate slopes and ridges characterised by hummocky debris lobes, often without clearly identifiable head scarps. Geomorphology reflects deep soil creep of the most weathered deposits.
Typical Composition, Weathering/ Layering and Variability	Hard silts and clays in red, brown, green, black, and purple with clasts of weathered extremely to very weak siltstone and sandstone. Commonly highly sheared matrix between clasts. Weathered transitional zone difficult to distinguish from underlying, less weathered rock mass, identifiable by subtle changes in moisture content and strength of clasts. Sheared matrix predominately silts and clays.
Behaviour, Instability Mechanism(s) and Principal Potential Geohazards	High plasticity clays prone to debris sliding and deep-seated creep, even on gentle slopes. Soil creep typically at a consistent weathering depth resulting in a predominantly planar movement morphology. Groundwater seepages are common from mid-slope. Swampy ground regularly observed in elevated areas, likely due to High soil shrink/ swell characteristics resulting in season soil expansivity.

Figure 5: Regional Geology (GNS Science Web Geology Map)

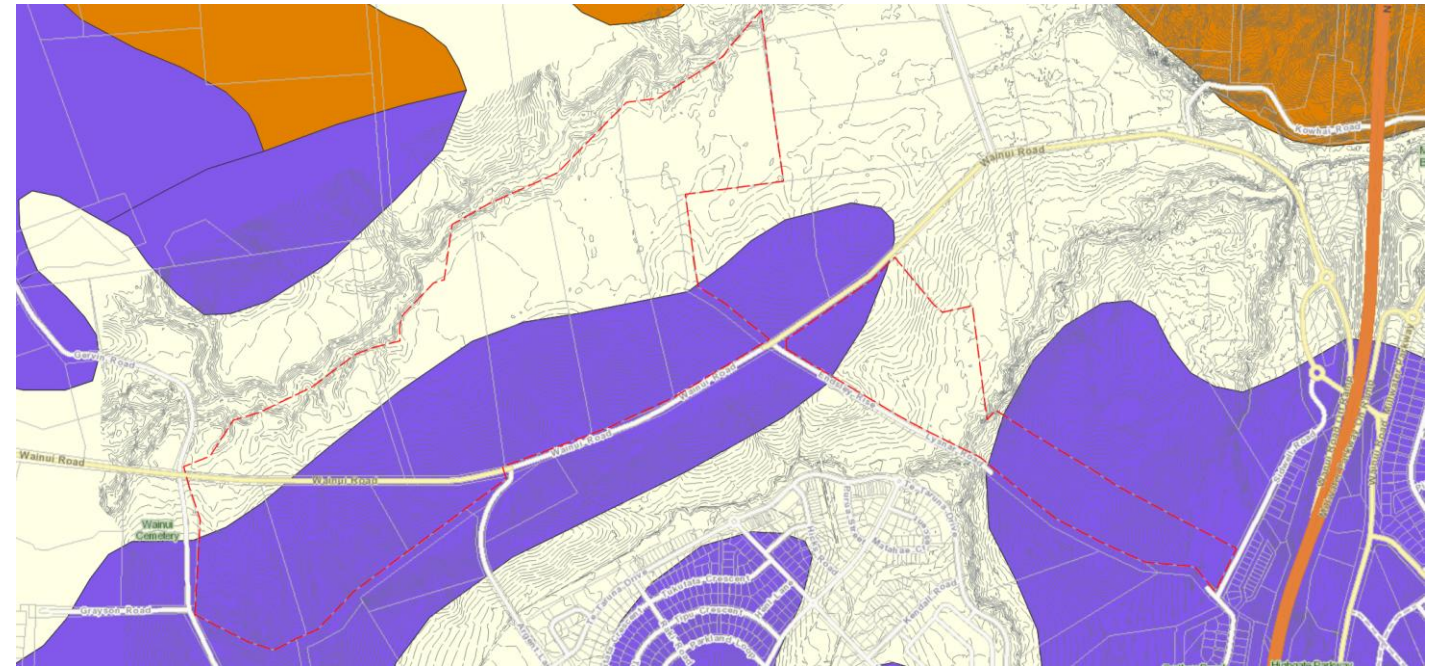


Figure 6: Hukerenui Mudstone



¹ Edbrooke, S.W. (compiler) 2001. Geology of the Auckland area. IGNS 1:250 000 geological map 3.

5.2 Geomorphology

Geomorphology of the site was mapped during a site walkover and in conjunction with examination of aerial photographs, and hill-shades. A geomorphology plan is presented in Figure 7 and appended Drawings 07 to 12, key notes are bullet pointed below:

Cemetery and Wainui Road North Blocks (Stage 14 ,15)

- A ridge forms the high point of the site along Wainui Road and the southern boundary of the proposed development
- Several gully / watercourses run from south to north from the Wainui road high point towards the Orewa Awa which bounds the site to the north
- From Wainui Road the site falls moderately ($<10^\circ$) to the north, a subtle break in slope is noted mid slope where the topography becomes gentler, at the base of the slope a large flat alluvial flood plain is present bound by the Orewa Awa. See Figure 8b.
- The Orewa Awa runs from the south west to the north east, parallel to the Waterloo Creek to the south and the Wainui Road Ridge.
- Within the Cemetery Road block, hummocky ground is noted on the northern slopes below the Wainui Road Ridge. These are shown in Figure 8a.
- Drainage typically runs south to north from the Wainui Road Ridge to the Orewa Awa, it has been modified by several farm ditches, partially in the flat alluvial terrace directly south of the Orewa Awa.

Endsley / Lysnar Road Block (Stage 16,17)

- South of Wainui Road the Wainui Road Ridge falls moderately to gently to the south towards the Waterloo Stream.
- A subtle change of slope is observed midslope to gently sloping to the south
- At the base of the slope, an alluvial valley is present above the banks of the Waterloo Stream, there are several incised gullies
- Main drainage through this block is a gully system that runs approximately northwest to southeast
- To the east on the Endsley Rise block, the topography rises moderately to the southeast before reaching a summit with a broad near flat area.
- To the southeast of the summit, the slopes fall moderately towards Sidwell Road. At the base of the slope a broad valley on Sidwell Road.
- Evidence of slope instability is noted on the southeastern slopes, with hummocky ground on the upper slopes and debris mounds at the base.

Figure 7: Geomorphology Plan

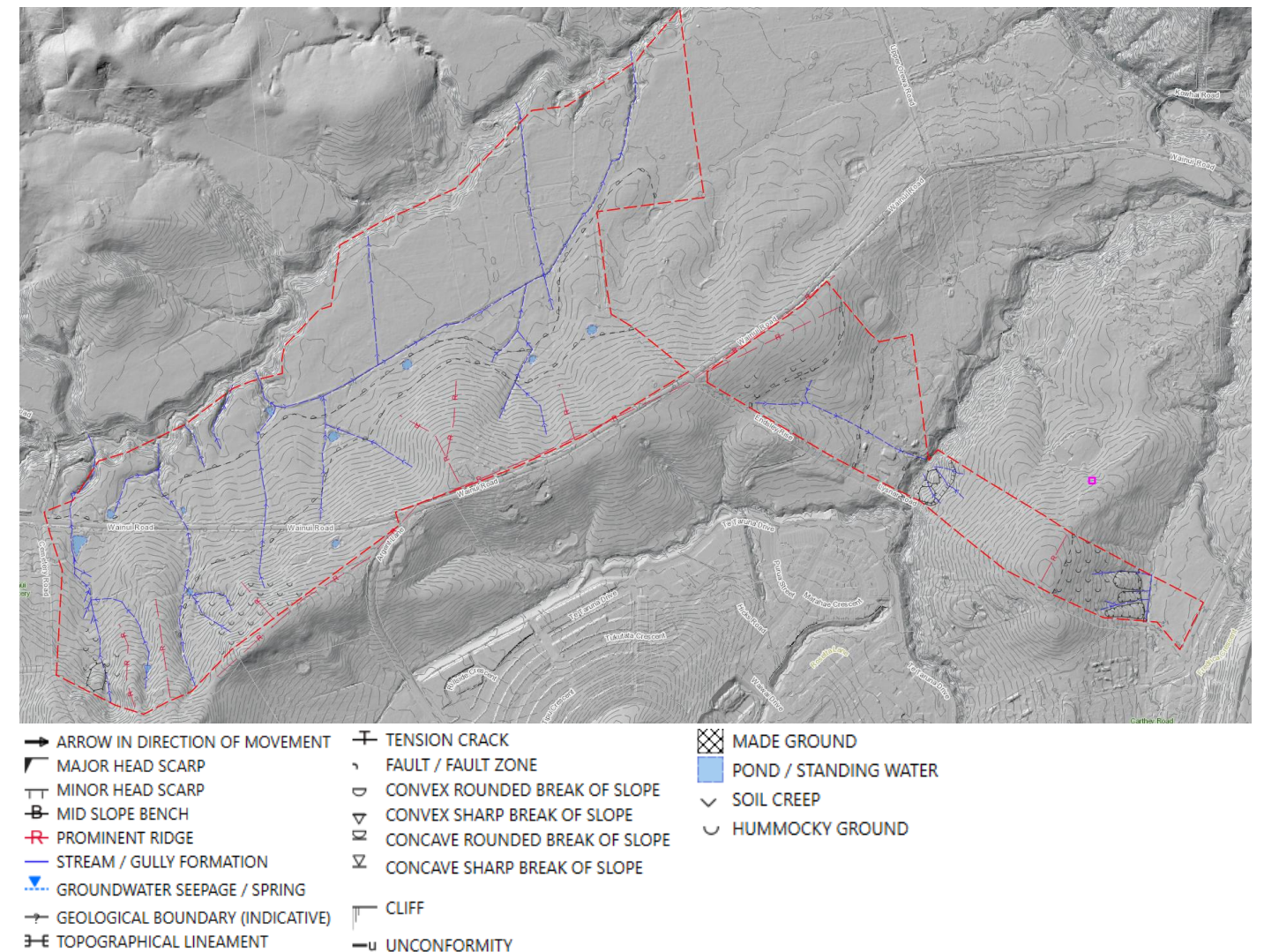


Figure 8a Left hummocky ground near Cemetery Road, 8b right alluvial plains south of the Orewa River



6 INVESTIGATIONS AND RESULTS

6.1 Field Investigation

The scope of fieldwork completed is shown below:

Table 3: Investigation Summary

Test IDs	Test Type	No. of Tests	Depth Range (m)
TPXX-26	Test Pits (TP)	55	4 - 5.2
MHXX-26	Machine Boreholes	5	15.3 - 19.88
CPTXX-26	Cone Penetration Tests	33	8.52 – 20.0

Engineering logs of the relevant investigations are presented in Appendix B.

The approximate locations of the respective investigation sites referred to above are shown on the Geotechnical Investigation Plan set as Drawings 01 – 06. Existing site investigations undertaken by others are also shown on these and are provided in Appendix C.

6.2 Laboratory Testing

All testing was scheduled by GRGE and carried out by Babbage Ltd an IANZ registered Testing Authority. The extent of testing carried out is presented below. Certificates for the test results are presented in Appendix D.

Table 4: Laboratory Testing

Test ID/ Location	Type of Tests	Test Method	Depth (m bgl)
MH03-26	Atterberg Limits, 1D Consolidation	NZS4402: 1986: Test 2.1 to 2.4 NZS4402: 1986: Test 7.1	2.5 – 3.0
MH03-26	Atterberg Limits, 1D Consolidation	NZS4402: 1986: Test 2.1 to 2.4 NZS4402: 1986: Test 7.1	5.5 – 6.0
MH03-26	Atterberg Limits, 1D Consolidation	NZS4402: 1986: Test 2.1 to 2.4 NZS4402: 1986: Test 7.1	8.5 – 9.0

6.3 Groundwater

During the investigation, completed in late summer conditions (February to March 2026), groundwater was encountered at depths ranging from 0.4m to greater than 5.0m below existing ground levels.

Within a number of the boreholes, groundwater monitoring piezometers were installed to various depths below existing ground level using 32mm uPVC, gravel and bentonite plug fitted with a toby cover. Results of groundwater monitoring undertaken following the investigation is presented below.

Table 5: Piezometer Details

Test ID	Screen (mbgl)	Depth	Screened Formation	Soil	13 March 2026	5 June 2026	30 June 2026	20 July 2027
					Depth (mbgl)			
MH01-26	9.5 – 14.5		Hukerenui Transitional		0.19	0.99	0.95	1.4
MH02-26	5 - 15		Alluvium		5.8	3.67	3.7	3.67
MH05-26	3 – 10		Hukerenui Transitional		1.0m	-	-	-

Figure 9 Site Investigation (GRGE)

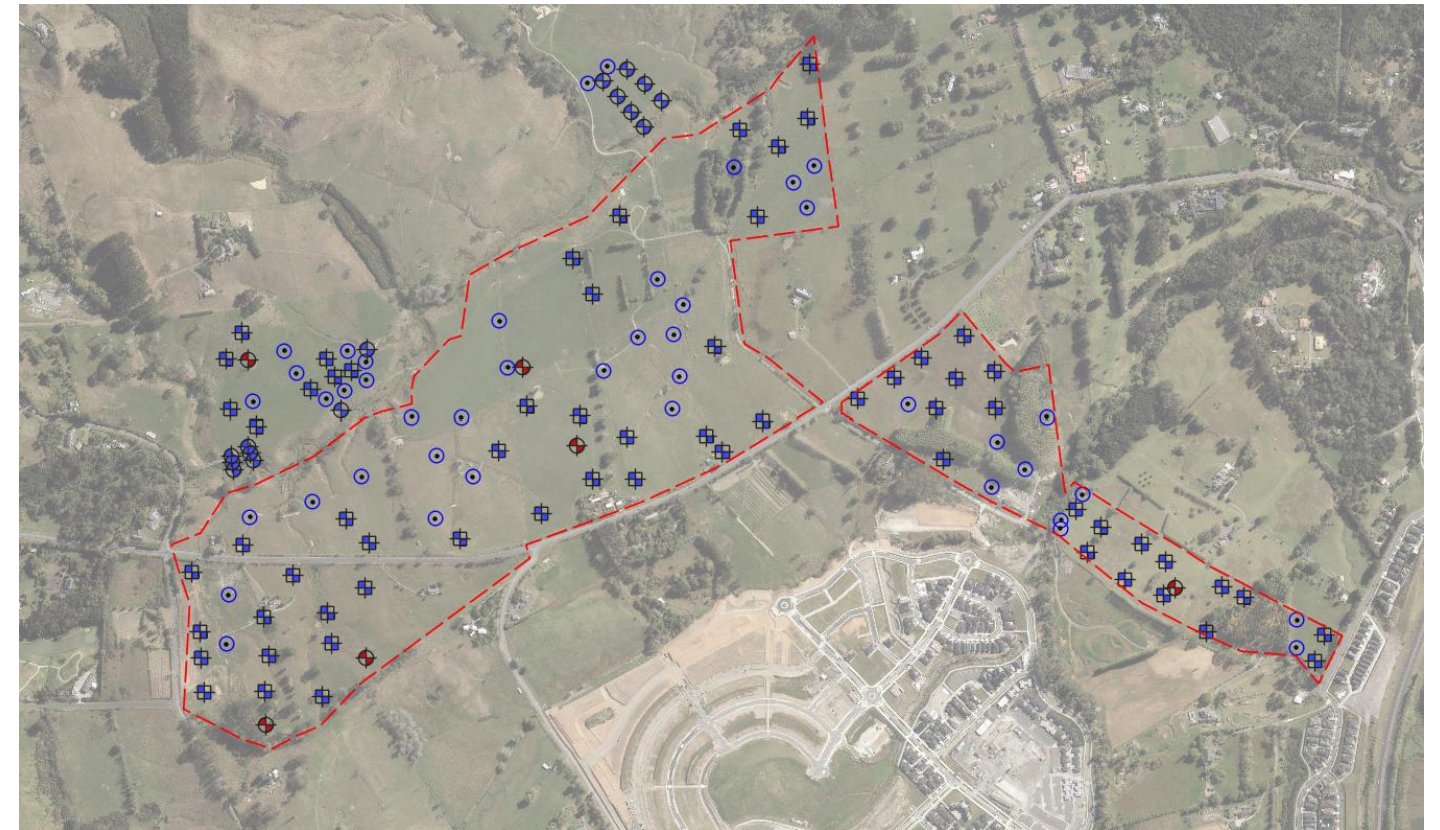
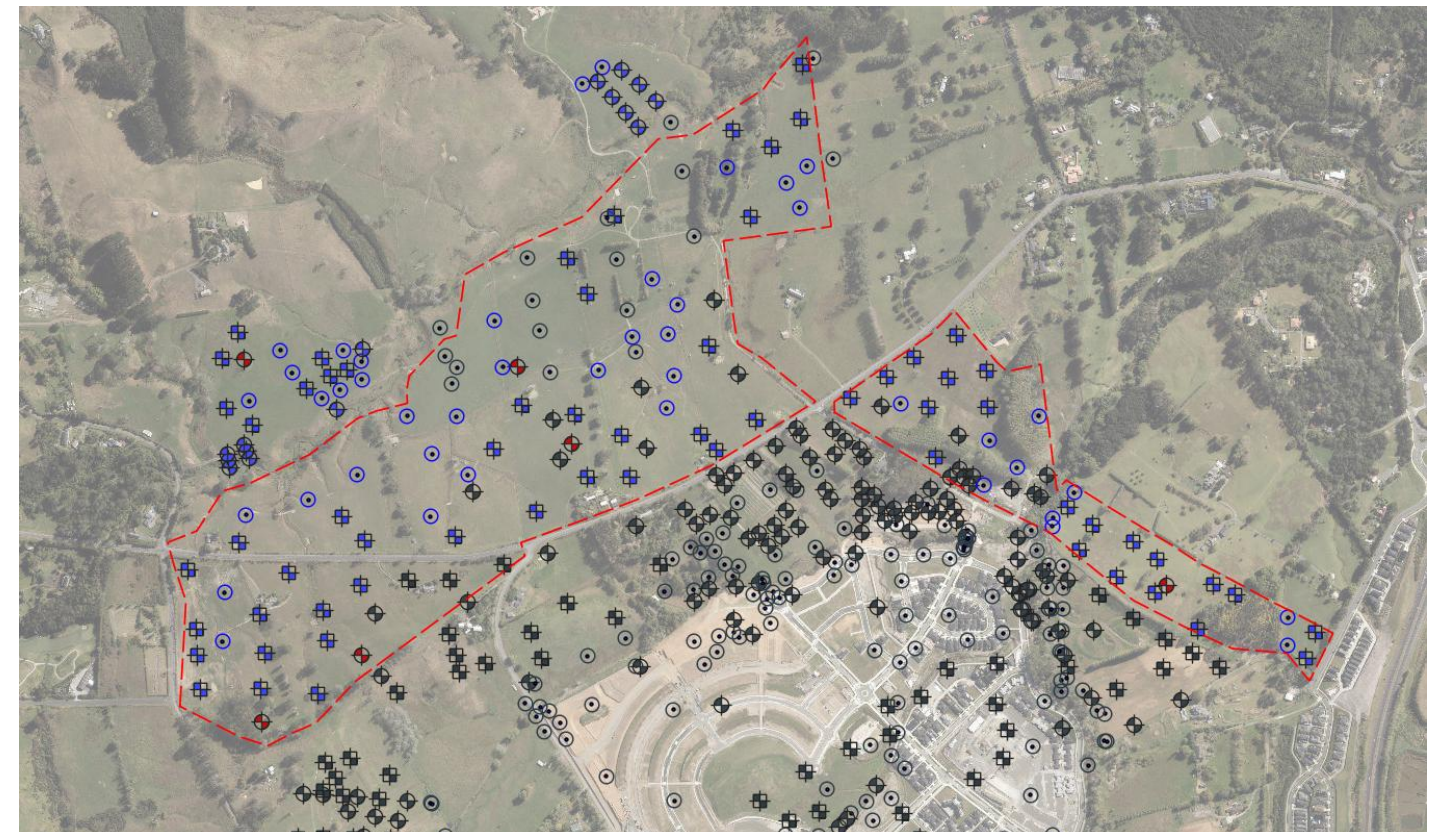


Figure 10 Existing Site Data



7 **GROUND MODEL**

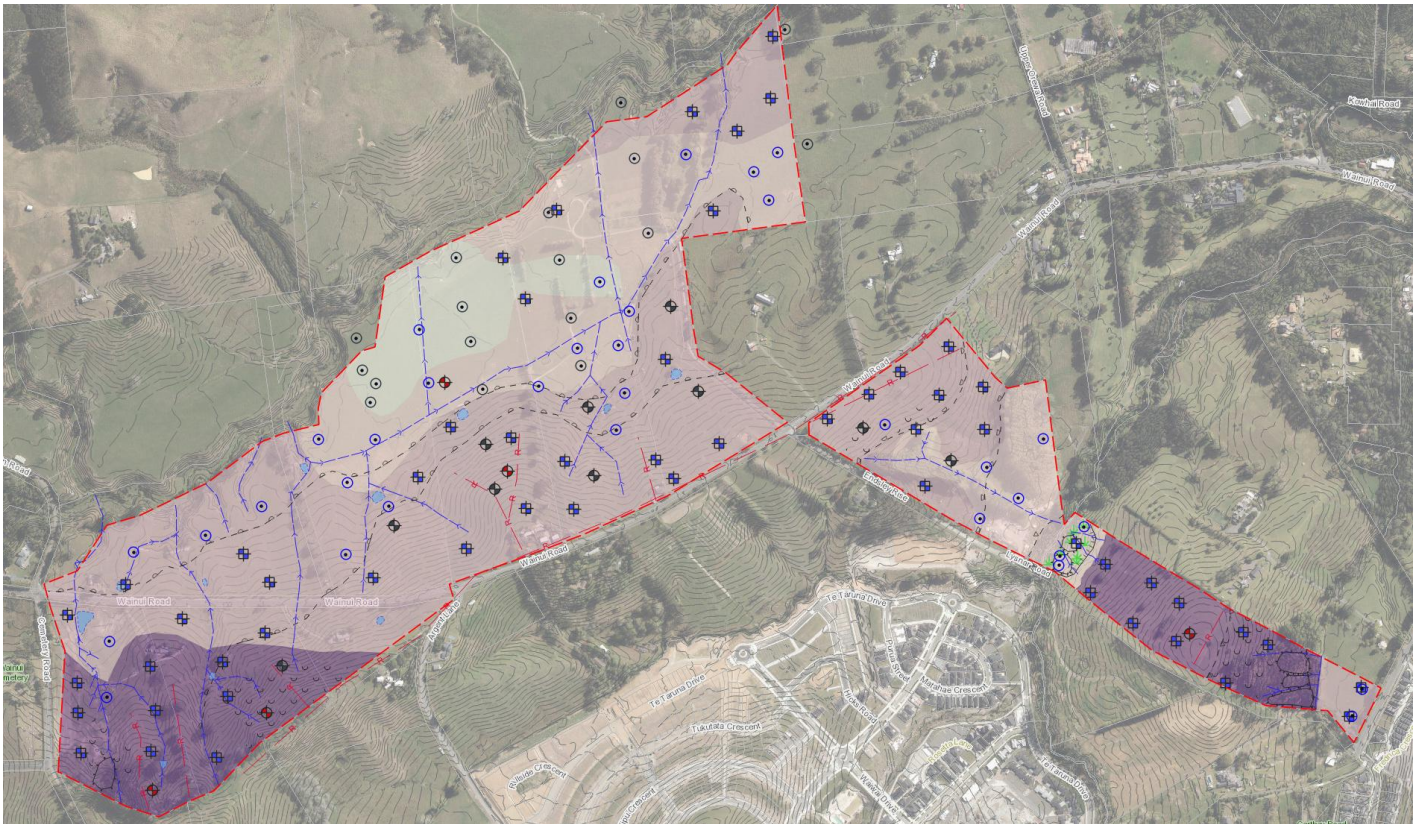
7.1 **Encountered Geology and Stratigraphy**

An interpretation of the local geology based on the ground investigation is provided in the following table including material description, geological history and geomorphology and on the combined geology and geomorphology Drawings 07 to 12. Our assessment of the distribution of the stratigraphic layers is illustrated on the appended Geological Sections A to M on Drawings 13 to 25.

Table 6: Ground Model

Geology (Map Code)	Geotechnical Unit	Material Description	Geomorphology and History	Key Geotechnical Hazard
Holocene Alluvium (Q1a)	Soft Alluvium	Soft highly plastic clay. Some organics, fibrous wood fragments. Some bands of fine to moderate sand.	Low lying flood plain near a significant mender of the Orewa River.	Soft and unconsolidated, low bearing capacity, excessive static settlement susceptibility. Potential for liquefaction in sandy layers.
Middle to Late Pleistocene Hill and River Deposits (IQa)	Firm Alluvium	Firm low to moderate plasticity clays and silts. Some organics (wood fragments)	Elevated areas of alluvial soils likely deposited when sea levels were much higher. Erosive surface incised by the Orewa River and Waterloo Stream. Firm alluvium situated on lower slopes.	Soils expected to be over consolidated due to previous loading. Soils compressible under high fill / building loading. Firm alluvium have low bearing strength. Liquefaction unlikely due to age of deposits.
	Stiff to Very Stiff Alluvium	Stiff to very stiff low plasticity clays and silts.		
Hukerenui Mudstone Northland Allochthon	Residual Soil	Stiff plastic clays and silts. Typically wet.	Situated on the most elevated slopes on site. Typically gentle to moderate slopes, hummocky ground in areas of steeper slopes. Some debris lobes noted at the base of slopes.	Potential for highly expansive soils.
	Transitional Soil	Stiff silts and clays with clasts of extremely weak highly weathered siltstone and sandstone clasts. Relic sheared fabric noted within soil mass. Moist to wet.		Prone to slope instability when exposed in cuts or overloaded with fills. Likely to rapidly degrade and soften if exposed to weather.
	Rockmass	Moderately to slightly weathered extremely to very weak siltstone and sandstone. Highly sheared with some slickenside surfaces on defects. Typically recovered as a silty gravel, typically dry.	Material highly sheared due to depositional environmental.	Likely to rapidly weather and soften when exposed.
East Coast Bays Formation Waitemata Group	ECBF Residual Soils	Stiff sandy clays and silts.	Fault controlled contact, not exposed at surface within the project area.	Potential for highly expansive soils. Bedding shears can be present near rock transition may cause instability when exposed.
	ECBF Rock Mass	Extremely to very weak siltstone and sandstone.		Rock mass

Figure 11: Geological map, legend is shown by unit colours in Table 6



7.2 Recommended Geotechnical Parameters

Parameter derivation can be found in Appendix E.

Table 7: Geotechnical Design Parameters

	Testing Results			Unit Weight	Drained		Undrained	Compressibility			
Geotechnical Unit	SPT N Value Range	Shear Vane Strength Range (kPa)	CPT Cone value (Qc)	γ (kN/m ³)	Drained Cohesion c' (kPa)	Drained Friction Angle ϕ' (deg)	Undrained Shear Strength S_u (kPa)	Constrained Modulus CPT M_{CPT}	Compression Index (Cc)	Initial void ratio (e ₀)	Coefficient of vertical consolidation (C _v) m ² /year
Soft Alluvium	0	15- 20 (18)	0.3 - 0.6	16	1	25	20*	0.5 - 3	0.63	1.75	15
Firm Alluvium	0 - 2	40 – 50 (41)	0.6 - 1.0	17	3	25	38*	3 - 6	0.25 - 0.4	0.86 – 1.2	5
Stiff to Very Stiff Alluvium	2- 5	50 – 200 (75)	1 – 2	17	3	28	65	6 - 8	No consolidation tests undertaken		
Residual Soil	3 – 7	60 - 200(100)	2 - 5	18	5	28	80	20 - 40			
Transitional Soil	11-30	90 - 200 (150)	5 – 10	18	10	12	100	40 - 150			
Rockmass	30 - 50	UTP	>15	20	20	28	300	-			
ECBF Residual Soils	-	131 – 154		18	5	30	80	-			
ECBF Rock Mass	-	-		20	30	40	1000	-			

Notes:

γ = soil unit weight (conservative value determined from CPT correlations and typical published values for similar soil types)

c' = effective cohesion (conservative industry accepted value)

ϕ' = effective friction angle (conservative industry accepted value, CMW laboratory testing values, CPT correlations)

S_u = undrained shear strength (lower bound value determined from vane shear testing and CPT correlations (NKT = 15) – refer correlation plots in Appendix E)

M_{CPT} = constrained modulus from CPT correlations.

Cc, Cr, e₀, C_v = consolidation parameters from laboratory testing where available.

*Typically adopted from CPT correlations

8 GEOHAZARDS ASSESSMENT

8.1 Risk Assessment

A full risk assessment in accordance with ACCOPs can be found in Appendix F. From this assessment the key geohazards for this project are assessed as:

- Static settlement of compressible alluvial soils due to fill and building loads
- Liquefaction and lateral spread of loose recent alluvial depths near the Orewa Awa
- Slope instability of Northland Allochthon slopes and earthworks batters
- Static settlement due to uncertified fill deposits such as tip sites
- Site wide expansive soils damaging future foundations

The following section provides further detail, including analysis of these hazards.

8.1.1 National Policy Statement on Natural Hazards

The following risk assessment has been produced in accordance with the National Policy Statement for natural hazards 2025, published December 2025, the likelihood, consequence and risk levels are referenced within this document.

Table 8 Risk Assessment

Hazard	Likelihood	Consequence	Unmitigated Risk	Comment
Flooding	Assessed by others			
Landslip	Likely	Moderate	High	Refer to Section 8.5 for assessment and Section 9.2 for mitigations.
Coastal erosion	Very Rare	Moderate	Low	The site is not near the coast
Active Faults	Very Rare	Moderate	Low	The site is not near any active faults
Liquefaction	Rare	Minor	Low	Refer Section 8.3 for detailed assessment.
Tsunami	Assessed by others			

8.2 Seismicity

A seismic assessment has been carried out in general accordance with NZGS guidance² to calculate the peak horizontal ground acceleration or PGA (amax) as follows:

$$a_{max} = C_{0,1000} \frac{R}{1.3} x f x g$$

Where: $C_{0,1000}$ = unweighted PGA coefficient (subsoil class C, D)

R = return period factor given in NZS1170.5, Table 3.5 (importance level 2)

f = site response factor subject to subsoil class (subsoil class C, D)

g = acceleration due to gravity

The ULS PGA was calculated based on a 50-year design life in accordance with the New Zealand Building Code³ and importance level (IL) 2 structures. The PGA for the serviceability limit state (SLS) and ultimate limit state (ULS) earthquake scenarios is as follows:

Table 9 Seismic Acceleration Parameters

Limit State	AEP	R	PGA(g)	Magnitude
SLS	1/25	0.25	0.06	5.9
ULS	1/500	1.0	0.19	5.9

Note: SLS = serviceability limit state; ULS = ultimate limit state AEP = annual exceedance probability

8.3 Liquefaction

General performance levels for liquefied deposits are presented below (as obtained from MBIE Module 3).

Figure 12: MBIE Module 3 Liquefaction Performance Levels

PERFORMANCE LEVEL	EFFECTS FROM EXCESS PORE WATER PRESSURE AND LIQUEFACTION	CHARACTERISTICS OF LIQUEFACTION AND ITS CONSEQUENCES	CHARACTERISTIC F_L , LPI
L0	Insignificant	No significant excess pore water pressures (no liquefaction).	$F_L > 1.4$ LPI=0 LSN <10
L1	Mild	Limited excess pore water pressures; negligible deformation of the ground and small settlements.	$F_L > 1.2$ LPI = 0 LSN = 5 – 15
L2	Moderate	Liquefaction occurs in layers of limited thickness (small proportion of the deposit, say 10 percent or less) and lateral extent; ground deformation results relatively small in differential settlements.	$F_L \approx 1.0$ LPI < 5 LSN 10 – 25
L3	High	Liquefaction occurs in significant portion of the deposit (say 30 percent to 50 percent) resulting in transient lateral displacements, moderate differential movements, and settlement of the ground in the order of 100mm to 200mm.	$F_L < 1.0$ LPI = 5 – 15 LSN = 15 – 35
L4	Severe	Complete liquefaction develops in most of the deposit resulting in large lateral displacements of the ground, excessive differential settlements and total settlement of over 200mm.	$F_L << 1.0$ LPI > 15 LSN > 30
L5	Very severe	Liquefaction resulting in lateral spreading (flow), large permanent lateral ground displacements and/or significant ground distortion (lateral strains/stretch, vertical offsets and angular distortion).	

The liquefaction susceptibility of the soils at this site was assessed in accordance with MBIE/NZGS guidance⁴ based on the following assumptions:

- In accordance with MBIE/NZGS guidance¹ and in the absence of site-specific shear wave velocity measurements, no aging / strength gain factor has been applied.
- Soils are also classified with respect to their grain size and plasticity to assess liquefaction susceptibility. For this project, a cut-off threshold soil behaviour type index value (I_c) of 2.6 was used to distinguish between liquefiable ($I_c < 2.6$) and non-liquefiable ($I_c > 2.6$) soils.
- Specific liquefaction analyses were undertaken for an IL2 structure, using the software package CLiq using the Boulanger and Idriss (2014) method. The cyclic stress ratio (CSR), being a function of the earthquake magnitude

² NZ Geotechnical Society publication "Earthquake geotechnical engineering practice, Module 1: Overview of the standards", (March 2016)

³ Ministry of Business, Innovation and Employment (1992) NZ Building Code Handbook, Third Edition, Amendment 13 (effective from 14 February 2014)

⁴ Earthquake Geotechnical Engineering Practice, Module 3: Identification, assessment and mitigation of liquefaction hazards", (November 2021)

for the design return period event, was compared to the cyclic resistance ratio (CRR), being a function of the CPT cone resistance (q_c) and friction ratio (F_r).

- Free-field liquefaction induced settlements were determined in accordance with Zhang et al. (2002). With respect to liquefaction response, consideration was given to a 10m cut-off depth to estimate index settlements as per MBIE⁵ guidance (foundation technical categories). These were compared to liquefaction settlement estimates over the full depth range of the CPT's with a depth weighting factor ranging from 1 at the ground surface to 0 at 18m depth applied to the volumetric strains (e_v) in accordance with Cetin et al (2009)⁶.
- Thin-layer correction was applied based on the rate of change of the soil behaviour type index value (I_c) method

The worst-case results are shown in the table below, all CPTs were analysed however we have not reported the CPTs that estimated 0mm of liquefaction induced settlement. Based on the above results, the site is expected to perform well with negligible liquefaction induced settlements. In addition, even CPTs with potentially liquefiable layers under the ULS case, the thickness of the upper non-liquefiable crust predicts that liquefaction damage is very unlikely (Ishihara 1985).

As such it is concluded that the site is equivalent to Technical Category TC1 or Very Low Liquefaction Vulnerability. Outputs of the analysis can be found in Appendix G.

Table 10: Calculated Liquefaction Settlements

CPT No.	SLS Settlement (mm)	Total ² ULS Settlement (mm)	Index ¹ ULS Settlement (mm)	Depth to Liquefiable Layer (m)
CPT06-26	0	<5	0	4.0
CPT07-26	0	<55	<15	8.5
CPT13-26	0	<5	<5	9.5
CPT16-26	0	<10	<15	4.0
CPT18-26	0	<5	<5	7.0
CPT26-26	0	<10	<10	8.0
CPT27-26	0	<5	<5	8.0
CPT02-22	0	<5	0	16.0
CPT08-22	0	<5	0	13.0

Note: All settlement and depths based on existing ground profile
¹Index settlements are calculated based on the upper 10m of the soil profile using no depth weighting factor.
²Total ULS settlements are based on the full depth of the CPT trace with a depth weighting factor applied.
³Index settlements are for assessment of the site against the MBIE site Technical Category guidelines and are not comparable to the total ULS settlements.

8.4 Cyclic Softening

The fine-grained alluvium, while not liquefiable due to its high plasticity, may be susceptible to some strength loss, referred to as cyclic softening, during a ULS seismic event.

Cyclic softening analyses of those soils was carried out in accordance with Boulanger⁷ and Idriss⁸. This correlates earthquake magnitude to the estimated number of equivalent stress cycles (Figure 13) and then correlates number of cycles to a cyclic shear strength ratio (Figure 14).

Based on the above assessment, 9 stress cycles are estimated during the ULS M6.5 earthquake resulting in a worst-case cyclic shear strength of 80% of the peak shear strength. Reduced shear strengths have been considered for the slope stability analysis in Appendix H.

Figure 13: Relationship Between Earthquake Magnitude and Mean Number of Uniform Stress Cycles

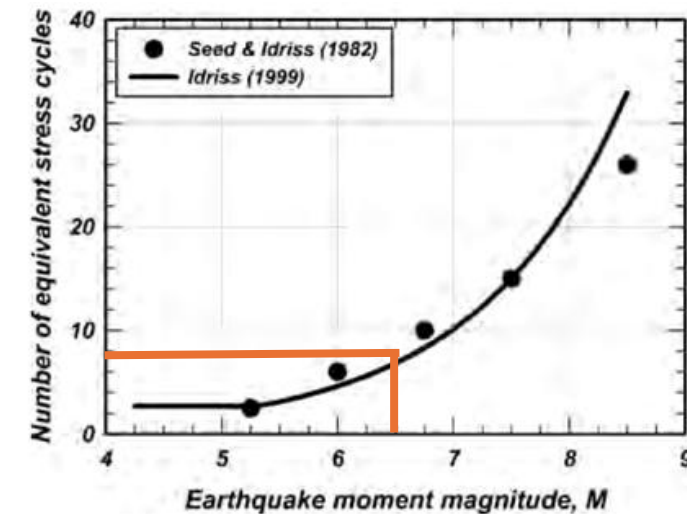
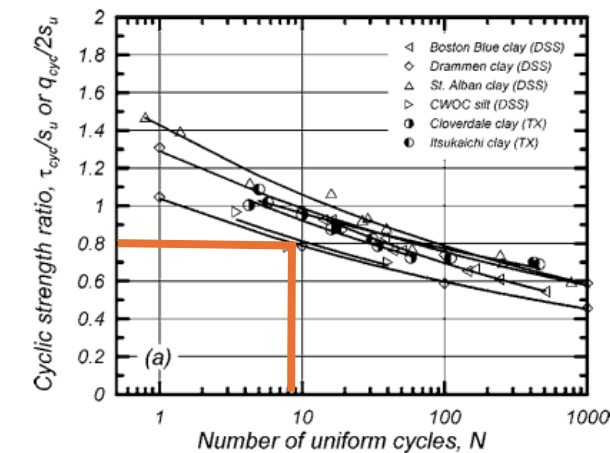


Figure 14: Relationship Between cyclic Strength Ratio and Number of Uniform Stress Cycles



⁵ Repairing and Rebuilding House affected by the Canterbury Earthquakes", (December 2012).

⁶ Cetin, K., Bilge, H., Wu, J., Kammerer, A., and Seed, R. (2009). Probabilistic Model for the Assessment of Cyclically Induced Reconsolidation (Volumetric) Settlements. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 135(3), pp. 387-398.

⁷ Boulanger, R.W. and Idriss, I. M. (2007) Evaluation of Cyclic Softening in Silts and Clays, Journal of Geotechnical and Environmental Engineering, Vol 133, Issue 6.

⁸ Idriss, I. M. and Boulanger, R. W. (2008) Soil Liquefaction During Earthquakes. Monograph 12, Earthquake Engineering Research Institute.

8.5 Slope Stability

Slope stability analysis has been carried out for 13 cross sections which were selected as areas of natural instability, steepest cut batters or largest fills or adverse geological conditions.

Slope stability analyses were undertaken using the Morgenstern-Price method of slices under translational failure mechanisms (generally being the critical mechanism) using the proprietary software SLIDE Version 6. Existing and proposed landforms were assessed.

The following performance standards were adopted, in accordance with Auckland Council Code of Practice for Land Development and Subdivision, Chapter 2

Table 8 Slope Stability Factor of Safety Performance Targets

Condition	Required Minimum Factor of Safety
Normal Groundwater Condition	1.5
Extreme (worst credible) groundwater condition	1.3
Seismic condition for ULS pga (calculated as 0.19g)	1.0*
Note*: Factor of safety < 1.0 acceptable where displacement-based approach demonstrates acceptable displacements	

Seismic displacements were estimated based on a Newmark Sliding Block approach using 50th percentile correlations published in Ambraseys & Srbulov⁹ and Jibson¹⁰. For the ULS event, the earthquake Magnitude 5.9 (M*) correlation was adopted. A 60km fault source distance (Wairoa North Fault) was also adopted for the Ambraseys & Srbulov correlations.

The following tables summarise the slope stability factors of safety achieved for both the existing and proposed landforms.

Table 92 Existing Section Slope Stability Factors of Safety

Section	Existing Slope Stability Minimum Factor of Safety		
	Normal GW	Extreme GW	ULS Seismic
A-A	2.9	2.4	2.7
B-B	2.7	2.4	2.3
C-C	1.6	1.4	1.2
D-D	1.3	1.2	2.0
E-E	1.4	0.9	1.5
F-F	2.0	1.8	1.3
G-G	2.5	1.8	0.8
H-H	2.5	2.1	1.2
I-I	1.7	1.8	1.0
K-K	2.3	2.3	1.5
L-L	2.2	1.7	1.3
M-M (NW Facing Slope)	1.6	1.5	3.3
M-M (SE Facing Slope)	3.6	2.7	4.0

Table 10 Proposed Sections Factor of Safety Requirements

Section	Proposed Slope Stability Minimum Factor of Safety				Seismic Yield Acceleration	ULS Displacement (mm)
	Normal GW	Extreme GW	ULS Seismic	With Cyclic Softening		
A-A	2.3	2.1	2.4			
B-B	1.2	0.7	1.4			
C-C	1.1	1.1	2.7			
D-D	1.2	1.1	2			
E-E	1.5	1.2	1.7			
F-F	1.6	1.4	1.1	3.1		
G-G	1.6	1.6	0.7*	1.1	0.08	20 - 35mm
H-H	2.1	1.7	1.2	2.8		
I-I	1.3	1.2	1.0	2.1		
K-K	2.3	2.4	1.1			
L-L	1.5	1.3	1.3			
M-M (NW facing slope)	1.4	1.2	1.8			
M-M (SE Facing Slope)	1.5	1.4	3.7			
* Likely to be conservative as it is expected that shear strength of soils will be higher at depth due to overburden pressure.						

Based on the above, slope stability factors of safety in accordance with ACCOP are not achieved for Sections B, C, D, I and M and therefore slope remedial works are required. Recommendations can be found in Section 9.2. Outputs can be found in Appendix H.

ULS seismic displacements on cross section G were estimated at 20-35mm, based on the building set back from the free face (Orewa River), lateral stretch on these buildings is expected to be minimal.

8.5.1 PC120 Auckland Unitary Plan Landslide Assessment

A landslide risk assessment in accordance with Appendix 24 of Plan Change 120 to the Auckland Unitary Plan (PC120) was carried out and is found in Appendix K. In summary:

- Large scale landslide susceptibility is mapped as very low to low
- Shallow landslide susceptibility is mapped as generally very low to moderate, with high to very high mapped near existing watercourses
- To determine run-out susceptibility, the highest landslide susceptibility class mapped within 150m of each stage was reviewed. This shows very high shallow landslide susceptibility for the site, in particular near water courses

The maps typically show very low to low shallow and large-scale landslide susceptibility. Where moderate to very high shallow landslide susceptibility exists, these are within areas designated for reserve and are well offset from future housing areas.

Based on the desktop assessment, we consider that landslide hazard result is Low (acceptable) under Appendix 24 of PC120.

⁹ Ambraseys, N. and Srbulov, M. (1995) Earthquake induced displacement of slopes. Journal of Soil Dynamics and Earthquake Engineering, Vol 14

¹⁰ Jibson, R.W., (2007) Regression models for estimating coseismic landslide displacement. Journal of Engineering Geology, Vol 91

8.6 Static Settlement

Proposed fill embankments and potential future building loads will induce settlements within the underlying subsoils. An initial screening exercise using CPeT-IT was undertaken to determine the areas of risk of static settlement. Oedometer testing was carried out within the soft alluvial soils to determine consolidation parameters to further understand the settlement risk associated with these soils.

8.6.1 CPT Method

Proposed fill embankments and potential future building loads will induce settlements within the underlying subsoils.

Preliminary load induced settlement analysis was undertaken using the software CPeT-IT, with primary settlements calculated according to the following formula:

$$S_p = \sum \frac{\Delta\sigma_v}{M_{CPT}} \Delta z$$

Where: $\Delta\sigma_v$ = change in effective stress
 M_{CPT} = constrained modulus from CPT
 Δz = change in depth

Secondary creep settlements were calculated according to the following equation:

$$S_c = C_\alpha \cdot \Delta z \cdot \log\left(\frac{t}{t_p}\right)$$

Where: C_α = coefficient of secondary compression
 t_p = duration of primary consolidation (12 months assumed)
 t = duration of design life (50 years)

These were used to undertake a screening of construction and post construction settlements, where:

- Construction settlements are the settlement induced by earthworks fill loads (primary only) at consolidation T90.
- Post construction settlements are the sum of the remaining primary (10% of primary), settlement induced by development loads (i.e. buildings, roads) and secondary creep settlement.
- Based on this screening, any areas with post construction settlements greater than 50mm are deemed unacceptable and require mitigation. These are highlighted in the table below.

Table 11 CPT Settlement Estimations

CPT No.	Compressible soil thickness (m)	Proposed Fill Height (m)	Construction Settlement, T90 (mm)	Post Construction Settlement (mm)
CPT01-26	4	1	15	20
CPT02-26	8	4	120	40
CPT03-26	4	1	30	60
CPT04-26	9	3	110	30
CPT05-26	12	3	630	340
CPT06-26	7.5	2	50	30
CPT07-26	8.5	1	40	30
CPT08-26	10.5	1	50	70
CPT09-26	12.5	1	60	90
CPT10-26	5	3	90	30
CPT12-26	8	2	70	50
CPT13-26	7	4	110	40

CPT No.	Compressible soil thickness (m)	Proposed Fill Height (m)	Construction Settlement, T90 (mm)	Post Construction Settlement (mm)
CPT14-26	6	2	50	20
CPT15-26	5	3	70	30
CPT16-26	10	3	130	60
CPT17-26	3	1	10	<5
CPT18-26	7	2	150	160
CPT19-26	9.5	2	90	70
CPT20-26	9	2	90	110
CPT21-26	12	1	180	330
CPT22-26	6	2	20	10
CPT23-26	6	0	0	40
CPT24-26	5	1	20	25
CPT25-26	2	5	70	20
CPT26-26	12	3	270	200
CPT27-26	8	1	160	280
CPT29-26	4	1	10	10
CPT30-26	5	3	230	80
CPT31-26	3	3	40	10
CPT32-26	9	3	220	110
CPT34-26	4	3	50	50
CPT35-26	5	2	50	40
CPT36-26	3	1	20	<5
CPT02-22	10	0	0	70
CPT03-22	7.5	1	120	130
CPT04-22	14.5	0	0	50
CPT05-22	13	2	170	140
CPT06-22	13.5	2	220	190
CPT08-22	12.5	1	60	80
CPT09-22	9.5	1	100	80
CPT10-22	12	2	200	180
CPT14-22	10	0	50	65
CPT15-22	9	1	90	60

Excessive post construction settlements are highlighted in red above, these exist in CPTs within the soft to firm alluvium geotechnical unit. Settlements range vastly within the soft to firm area with post construction settlements ranging from 200mm to 340mm. CPT-IT outputs can be found in Appendix I. Further analysis of these soils are carried out below.

8.6.2 Soft and Firm Soils 1D Consolidation Settlement Assessment

Oedometer laboratory testing was also carried out to assess the soil modulus parameters for load induced settlement analyses of the soft and firm alluvial soils.

Results of the laboratory oedometer testing show that the soils are near-normally consolidated meaning that for the proposed fill embankment, these soils will experience significant consolidation settlement following the virgin compression (Cc) line. Compressibility parameters based on the limited oedometer testing can be found in Section 7.2.

8.6.3 Settlement Predictions

The construction of earthfill embankments over the soft and firm alluvial soils of the Orewa River Flood Plain will induce consolidation and post construction creep settlements. An assessment of static settlements was completed for the proposed range of fill embankment heights.

Primary consolidation settlements were assessed in accordance with the following Terzaghi 1-dimensional consolidation theory for normally consolidated soils by spreadsheet and Settle 3 software.

$$S_{consol} = \frac{C_c}{1 + e_0} \cdot H \cdot \log\left(\frac{\sigma'_v + \Delta\sigma_v}{\sigma'_v}\right)$$

Where S_{consol} = consolidation settlement, C_c and e₀ are defined in Table 7 above, H = thickness of compressible layer (assumed to be 10m as an average thickness), σ'_v = initial vertical stress and Δσ_v = change in vertical stress or load applied. In construction practice, 90% of the consolidation settlement (t₉₀) is often targeted during the construction phase as noted above.

Creep settlements (S_{creep}) were estimated in accordance with the following relationship:

$$S_{creep} = \frac{C_\alpha}{1 + e_0} \cdot H \cdot \log\left(\frac{t}{t_l}\right)$$

Where C_α and e₀ are defined in Table 7 above, H = thickness of compressible layer, t = design life (50 years), t_l = t₉₀ or construction period, whichever is greatest.

Estimated static settlements are summarised as follows for an average 10m soil profile.

Table 12 Consolidation Settlement Estimations

Unit	Compressible soil thickness (m)	Proposed Fill Height (m)	Construction Settlement, T90 (mm)	Post Construction Settlement (mm)
Soft Alluvium	10	0 (building loads only)	0	350 (building only)
		1	360	320
		2	600	330
Firm Alluvium		0 (building load only)	0	240 – 320 (building only)
		1	210 – 280	150 – 220
		2	450 - 620	150 – 200
		3	580 - 780	130 - 160

It must be noted that the calculations above assume a worst-case soil profile comprising continuous soft or firm soils of 10m, in reality, stiffer bands exist within the soft/ firm soil profile, these results are likely upper bound.

This reflected by the results of the CPT construction settlement estimates, which in most cases are approximately half of the table above.

These results further highlight that predicted post construction ground settlements are considered excessive for the proposed development where specific ground improvement measures will be required, as discussed in Section 8.8.1.

8.6.4 Time Rate of Settlement

The rate of consolidation settlement is dictated by the rate in which excess pore water pressures can dissipate is dictated by its permeability and thickness. The ground model presents assumes an average compressible silt / clay layer thickness

of 10m. In most cases, the compressible layer is underlain by low permeability silt / clay / rock that will present only 1-way drainage.

Time rates of settlement are notoriously difficult to estimate due to their depositional environment where there is inherent material composition lateral and depth variability and presence of intermediate sand lenses. Reference to coefficient of vertical consolidation (C_v) laboratory data shows a wide scatter of data although a value of 5m²/year of the firm alluvium and 15m²/year for the soft alluvium looks to provide a reasonable lower bound estimate for preliminary design purposes.

Based on those values, the following settlement timeframes are estimated:

Table 13 Time Rates of Settlement

Soil Layer Thickness (m)	Hdr (m)	Time to Reach t90 (years)	
		Soft	Firm
5	5	2	3
10	10	5	5
15	15	10	10
Note: Hdr = maximum drainage path length			

It is noted that C_v values measured in the lab represent a particularly small volume of soil and can therefore be conservative as they don't take into account thin sand lenses and other discontinuities within the soil mass. Actual settlement timeframes may therefore be less than those tabulated above.

9 RECOMMENDATIONS

9.1 Site Suitability

Based on the assessment undertaken in this report, the following recommendations are made to avoid, remedy or mitigate potential adverse effects associated with the proposal and to support the appropriate implementation of the development.

In general, the proposal area is suitable for the development activity, however there are several areas of the site that will require land remediation during the earthworks to reduce the effects of geotechnical hazards to acceptable levels. These are highlighted below:

- Slope remediation where steep (1V:2H) Northland Allochthon batter slopes are proposed
- Development over soft and firm alluvial soils near the Orewa River and Waterloo Creek

The following sections provide recommendations for mitigation of these hazards, using typical methods used in previous stages of the wider Milldale Development, or the wider North Auckland region.

Remedial measures are shown on the Remediation Plans Drawings 26 to 31 in Appendix A, with details in the following drawings 32 to 34.

Provided these recommendations are followed, the residual risk to natural hazards listed in Table 8 in respect to the National Policy Statement on Natural Hazards can be considered low.

9.2 Slope Stability Management

Results of the slope stability analyses discussed in Section 8.5 demonstrate that design landform gradients through portions the proposed development will not achieve the requisite slope stability factors of safety. The following preliminary remedial works have been designed:

- Shear keys where Northland Allochthon transition zone is present near the surface (detail on drawing 32);
- Geogrids within the proposed 1V:2H batter slopes to form reinforced earth batter slopes, both cut and fill slopes will require geogrids to manage long term slope creep, refer to drawing 32 for typical detail;
- Over excavation to allow for the placement of geogrid reinforced engineered fill where Northland Allochthon Transition Zone (detail on drawing 32). The buttress width is the width in m beyond the typical RE slope geogrid detail.
- Groundwater control (underfill drain plans and details on drawings 26 to 31 with detail on 34)

The resultant slope stability factors of safety are displayed in the Table below.

Table 14 Remediated Case Slope Stability Factors of Safety

Section	Remediated Slope Stability Minimum Factor of Safety			Remedial works
	Normal GW	Extreme GW	ULS Seismic	
B-B	1.5	1.4	1.4	Shear Key - SK01 and Retaining Wall
C-C	1.5	1.3	2.7	Shear Key / RE Slope (Fill Buttress) -SK02
D-D	1.5	1.3	2.0	
E-E	1.6	1.3	1.7	Shear Key – SK04
I-I	1.5	1.4	1.0	Shear key – SK05
M-M (NW facing slope)	1.5	1.3	2.1	Shear Key SK03 and subsoil drainage, piled retaining wall.

The extent of the various slope remedial works is shown on Drawings 26 to 31 in Appendix A. Slope remedial measures are summarised below per area.

Cemetery Road Block, Stage 15

Proposed 1V:2H reinforced earth slope in cut near Wainui Road will expose the Transitional Hukerenui Mudstone near the Wainui Road, as a result, slope remediation is required which is shown on slope stability sections B, C and D. The proposed remediation here is to excavate out the Transitional Hukerenui Mudstone to create an engineered fill shear key

founded within the Hukerenui Rockmass and for Sections C and D create an engineered fill buttress behind the reinforced earth slope to achieve global stability factors of safety.

Wainui Road North Block, Stage 14

Similar to Cemetery Road, where proposed reinforced earth batter slopes (1V:2H) expose the Transitional Hukerenui mudstone, shear keys are required to achieve the required factor of safety, in both Section E and I, the shear keys are typically shallow depth (5m).

In addition, care will be required when filling / surcharging or stockpiling on soft soil near the Orewa River and other watercourses due to short term instability caused by bearing capacity failures. This should be checked prior and during construction once staging is understood.

Lysnar Road / Endsley rise Block Stages 16, 17

Similar to Wainui Road North, no long term slope stability issues were identified on the Lysnar Road block, however there are soft soils near the Waterloo Stream which may be susceptible to bearing capacity failure again these should be checked prior and during construction once construction staging is understood.

The Endsley rise block requires both cutting and filling to form several platforms stepping up the existing slope on both hill faces, as well as filling on an existing landslip feature to the west. Shear keys are proposed beneath the existing landslip, along with subsoil drainage. Our current stability modelling does not suggest further shear keys are required beneath reinforced earth slopes, however there is potential for this following further investigation if significant shear zones are encountered within the Hukerenui Mudstone.

Retaining wall 16 (2.5-3.0m height) will require a slope stability analysis as part of detailed design as it will contribute to the global stability of the landform.

A schedule of shear keys and RE slope buttresses is shown in Table 18 below, with a section and detail on Drawing 33 and 34.

Table 15 Schedule of Shear Keys and Fill Buttress

Shear key reference	Base width (m)	Depth below lowest earthworks surface (m)	RE Slope fill buttress width (m)	Comments
SK01	6.0	2.0	-	
SK02	6.0 – 7.0	2.0 – 4.0	4.0	
SK03	8.0	6.0 - 7.0	-	Shear key also requires undercutting of firm alluvium. Shear key drainage required to outlet to the Waterloo Creek.
SK04	3.5	2.0	-	
SK05	5.0	2.0	-	

These details are preliminary only for the purpose of demonstrating that the slopes can be remediated using typical earthworks methods used elsewhere on the Milldale Project, to support the fast-track consent. Detailed design including additional geotechnical investigation is likely to be required ahead of each earthworks stage to further define the extent of these remedial works.

9.3 Settlement Mitigation

9.3.1 Ground Improvement Options

To mitigate post construction static ground settlements to acceptable development levels, the following options may be considered:

- Construction of a temporary surcharge or pre-load fill embankment above design finished level, with or without wick drains, to over-consolidate the compressible soils;
- Undertake ground improvement beneath the embankment footprint, such as stone columns, soil mixed columns, CFA piles or rigid inclusions to transfer loads from the embankment to more competent underlying soils at depth.

9.3.2 Ground Improvement Design

It is expected that pre-loading or surcharging is likely to be the preferred ground improvement technique for this project to reduce post construction static settlements to acceptable magnitudes. This method has been used extensively throughout the wider Milldale Project, in particular near the Waterloo Stream.

Preliminary pre-load designs were carried out using consolidation parameters noted in Section 7.

Resulting creep settlement magnitudes were estimated using the method described in Mesri et al (1994) based on the following:

- Weak alluvium zone – widespread floor loads = 12kPa (individual lots) or 20kPa (super lots), post construction settlement = 50mm;
- 90% consolidation (t_{90}) achieved during pre-load.

Based on those requirements, the following preliminary temporary surcharge embankment heights, above design building platform level are provided below, and should be used for earthworks planning purposes.

Table 16 Preliminary Preload Surcharge Heights

Unit	Compressible soil thickness (m)*	Proposed Fill Height (m)	Assume widespread development load (m)	Preliminary Surcharge Height (m)**
Soft Alluvium	9	0	12	1.7
		1.0		2.2
		2.0		2.7
Firm Alluvium		0		1.5
		1.0		2.0
		2.0		2.4
		3.0		2.7

* Assumes that a 1m undercut has been carried out and replaced with engineered fill to improve bearing capacity, refer to Section 9.4.2 for details.

** Surcharge is assumed to be constructed out of topsoil where $\gamma' = 16\text{kN/m}^3$. Denser material such as a clay fill or hardfill will reduce surcharge heights

Preliminary preload surcharge areas can be found on Drawing 26 to 31 in Appendix A. Where varying depths of compressible soils are present, preload surcharge heights will vary. Detailed design of the preload surcharges is required prior to each subdivision stage, including additional CPTs to optimise surcharge heights and locations.

Settlements due to preload surcharging are estimated to be between 200 and 900mm, and therefore it is expected that fill will need to be placed once the surcharge is removed to bring the site back to finished levels, or alternatively additional fill can be placed prior to surcharging.

Further analysis is required to determine preload requirements where alluvial soils are shallow over various fill heights, however preliminary analysis suggest that where both soft and firm alluvial soils are present ground improvement will be required where soft/firm soil thicknesses are greater than 4m.

9.3.3 Trial Embankment

A fully instrumented trial pre-load embankment constructed over the soft/firm alluvium area is strongly recommended prior to construction of the fill embankments to enable back-calculation of actual consolidation properties of the compressible soil unit.

This will provide valuable information to inform the design of the main embankment with respect to both fill volume allowances, construction program and temporary embankment stability. Adequate time must be available to enable a full assessment of settlement trends.

It is envisaged that a combination of the following devices would be implemented:

- Surface settlement plates, placed over the ground surface prior to filling, to assess total settlement magnitudes, which provide a cost effective robust monitoring technique;
- Vibrating wire piezometers installed at multiple depths to assess pore water pressure dissipation profiles for settlement purposes;
- Extensometers, particularly where more than one compressible layer is present, to gauge relative settlements in each unit.

A preliminary location of the trial embankment is shown on Drawing 29 in Appendix A.

9.3.4 Settlement Monitoring

There will inevitably be some variation in soil composition and resulting settlement profiles from one location to the next, it is imperative that settlement monitoring is undertaken during construction to back analyse consolidation settlement parameters and progressively refine the pre-load design.

Due to material composition and depth variability regular monitoring points will be required, preliminary locations are shown on Drawings 26 to 31 in Appendix A.

9.3.5 Existing Wastewater Main

The Milldale to Millwater Wastewater Main is present near the Waterloo Stream within the Lysnar Road / Endsley Rise block. This is a 900mm concrete pipe laid within compressible soil, which could be subject to differential settlement when fill and future building loads are applied.

Future loads above this pipe should be avoided, the following options are:

- avoiding all fill loads above the pipe, structural bridging required for buildings as per typical Watercare Standards
- bridging the fill by installation of ground improvement such as rigid inclusions or rammed aggregate piers with a load transfer platform to transfer vertical loads from the fill above to the underlying rock / soil mass.

9.4 Earthworks

All earthwork activities must be carried out in general accordance with the requirements of NZS 4431:2022¹¹ and the requirements of the Auckland Council Infrastructure Development Code under the guidance of a Chartered Geotechnical Professional. A Geotechnical Works Specification is provided as Appendix J and standard detail drawings are provided as Drawing 32 to Drawing 34 in Appendix A. Between them, these documents provide the requirements for site preparation, fill placement, subsoil drainage, compaction requirements, quality assurance observations and testing and as-built requirements.

A summary of the key project specific construction risks, as presented in the Project Risk Register in Appendix F are as follows:

Table 20: Key Construction Risks

Item	Mitigation Measures
Excavatability	Excavations likely to comprise alluvial soils or transitional Hukerenui mudstone which are able to excavated using typical earthworking plant. No evidence of harder limestone units found in previous stages.
Sediment Retention Ponds	Geotechnical Engineer to have input on the locations of these when situated on slopes. May require structural design including key and compaction specifications, subgrade inspections and QA of embankment materials.
Stockpile Locations	The locations including the heights must be approved by the Geotechnical Engineer. Issues in respect to slope instability occur on steep slopes and potential for bearing capacity failures on soft soils. Of particular risk is within

¹¹ Standards New Zealand (2022) Engineered fill construction for lightweight structures, NZS 4431:2022

	the 'soft to firm' alluvial soils near open watercourses, instrumentation may be required to monitor pore water pressure during filling.
Subgrade Preparation	Filling on the soft and firm alluvial soils may be problematic during periods of wet weather and with heavily laden plant. Undercutting may be required to create stiff subgrade base, refer to 9.4.2.
Service Trenches (trench collapse)	Unsupported trench collapse is likely in the soft and firm alluvial soils, particularly when beneath the groundwater table. The contractor must consider these soils as "complex" and consider trench support or battering in this area.
Land Instability During Works	In sloping ground consider: - Constructing shear keys early in the programme - Limiting the amount of shear key excavation open at any time, plan works around weather - Installing drainage prior to excavations. - monitor the works for signs of slope movement, alert geotechnical engineer if suspected.

9.4.1 Rock Mass Exposure

Where Hukerenui rock mass is exposed at subgrade, it is recommended that this is either undercut and replaced with engineered or alternatively ripped and recompacted to a depth of .75m below subgrade. This is to cap the rock mass to protect it from water ingress leading to slope instability, and rapid weathering causing softening and poor founding conditions. Likely areas are shown on the remediation plans (Drawings 26 to 31 in Appendix A)

9.4.2 Soft to Firm Alluvium Subgrade

Where lots are situated within the soft or firm alluvium at subgrade, bearing capacities will not meet 300kPa ultimate bearing capacity in accordance with NZS3604. To achieve this, we recommend that the upper 1m is undercut and replaced with engineered fill to create a raft of improved ground above the soft ground. This will also improve trafficking during earthworks and civil works.

Where preload surcharging occurs, we expect some stiffening of the soils at depth as a result of pore pressure dissipation.

9.4.3 Underfill Drainage

Underfill drainage is proposed to control groundwater levels during filling. Indicative underfill drainage locations can be found on Drawings 26 to 31 in Appendix A and a typical detail on Drawing 36.

9.4.4 Uncertified Fills

Where uncertified fills are identified, these must be assessed by the Geotechnical Engineer on a case by case basis. If unsuitable such as rubbish or organic deposits are encountered, these will need to be undercut and replaced with engineered fill. Possible locations are shown on the remediation Drawings 26-31 in Appendix A.

9.5 Civil Works

Table 17: Key Civil Inputs

Item	Value	Comment
Subgrade CBR	3-5% Residual and Stiff Alluvium 0-1% Firm and Soft Alluvium 5-6% Engineered Fill	To be confirmed during construction once subgrades exposed
Typical Topsoil Depths	200 – 1000mm	Refer to bore logs for detail on each stage.

Retaining Wall Parameters		Refer to Table 7 for soil parameters, must be reviewed by a Geotechnical Engineer for use.
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9.6 Foundations

Table 18: Anticipated Shallow Foundation Details

Item	Methodology	Value	Comment
Geotechnical Ultimate Bearing Capacity (GUBC) for shallow foundations		300 kPa	Available within stiff alluvium, residual northland allochthon and engineered fill at GCR stage. Soft to firm alluvium does not achieve 300kPa without further ground improvement as per recommendations section 9.4.2. Short axis of footing measuring no greater than 2.5m in plan
Expansive Soil Site Class	AS2870	M (moderate) to H (high)	Anticipated characteristic surface movement of up to 40 (M) to 60mm (H1), to be confirmed during each GCR stage by specific testing.
Strength reduction factors	B1/VM2 ¹²	0.8 0.5	Load combinations involving earthquake overstrength All other load combinations
Seismic Site Class(es)	NZS1170	C or D	Depending on location on site, to be provided on a lot by lot basis at GCR stage.

9.7 Certification

At the completion of the works, a Geotechnical Completion Report (GCR) will be prepared. The GCR will:

- report on the works undertaken
- confirm foundation design parameters
- describe future building and/ or earthworks limitations
- apply any restrictions that require further engineering investigation and/ or design on individual lots to avoid future building works exacerbating a natural hazard

10 FURTHER WORK

The following further work is recommended as the project progresses:

- Further geotechnical investigation on a stage by stage basis for specific design of retaining walls, shear keys, reinforced earth slopes and preload surcharge areas.
- Trial Embankment to define consolidation parameters for earthworks planning.

During construction, regular inspections and testing will be required as outlined in the Geotechnical Works Specification.

At the completion of the works, a Geotechnical Completion Report (GCR) will be prepared by a Chartered Professional Engineer outlining the works completed and all recommendations and restrictions for future development on each lot.

¹² Ministry of Business, Innovation and Employment (2025) Acceptable Solutions and Verification Methods for NZ Building Code Clause B1 Structure, B1/VM2, Second Edition

Gibb Ritchie Geotechnical Engineers has prepared this report for Fulton Hogan Land Development Ltd in accordance with the agreed scope of work for the Milldale Stages 14 - 17, Wainui East project. It provides geotechnical information and recommendations to assist in design and construction planning.

We view this report as part of an ongoing process. Our team can provide further input during detailed design and construction to help ensure that the recommendations are applied effectively and to provide advice should any unexpected ground conditions arise.

If you have any questions about the report, or if you would like to discuss the next steps for your project, please contact our team.

USING YOUR GIBB RITCHIE REPORT

Geotechnical reporting relies on interpretation of site information using experience, professional judgement, and opinion. As such, all geotechnical reports carry an inherent level of uncertainty, often greater than in other engineering disciplines. The notes below provide guidance on what can reasonably be expected from this report and its inherent limitations.

Purpose of this report

This report has been prepared specifically for Fulton Hogan Land Development Ltd and the Milldale Stages 14 - 17, Wainui East project. The contents may not meet the needs of others who may have different objectives or requirements. The report has been prepared using generally accepted geotechnical engineering and engineering geology practices. The opinions and conclusions are based on those principles.

Dependence on available information

This report is based on the information available at the time of preparation, including investigation results and referenced data. If further information becomes available or the project scope changes, the findings may no longer be valid. In such cases, Gibb Ritchie Geotechnical Engineers should be engaged to review and update the report.

Applicability to your project

The findings and recommendations in this report relate solely to the site and project described. They should not be applied to other sites or projects, as subsurface conditions and requirements differ between locations and developments. Similarly, applying data or conclusions from other sites to this project may lead to incorrect assumptions.

Interpretation of subsurface condition

Site investigations provide information at discrete locations. Conditions between or beyond these points may differ. Geological interfaces may be more variable than inferred, and actual conditions may only become clear during construction. Engineering judgement has been applied to interpret the available data, but no investigation can remove all uncertainty.

Changes in subsurface conditions

Ground and groundwater conditions can change over time, whether due to natural processes (seasonal variation, settlement, erosion) or human activities (earthworks, adjacent development). This report reflects conditions observed at the time of investigation. Confirmation of conditions may be necessary if significant time has passed before construction.

Role during construction

Recommendations in this report are based on selective point sampling and interpretation. It is strongly recommended that Gibb Ritchie Geotechnical Engineers remain involved during construction to confirm assumptions, review any changes in conditions, and provide timely advice if unexpected ground conditions are encountered.

Use by other parties

This report has been prepared for the named client. Where other design professionals or third parties intend to rely on it, they must first seek consent from Gibb Ritchie Geotechnical Engineers. Misinterpretation of geotechnical reports can lead to costly design or construction issues, and our continued involvement helps reduce these risks.

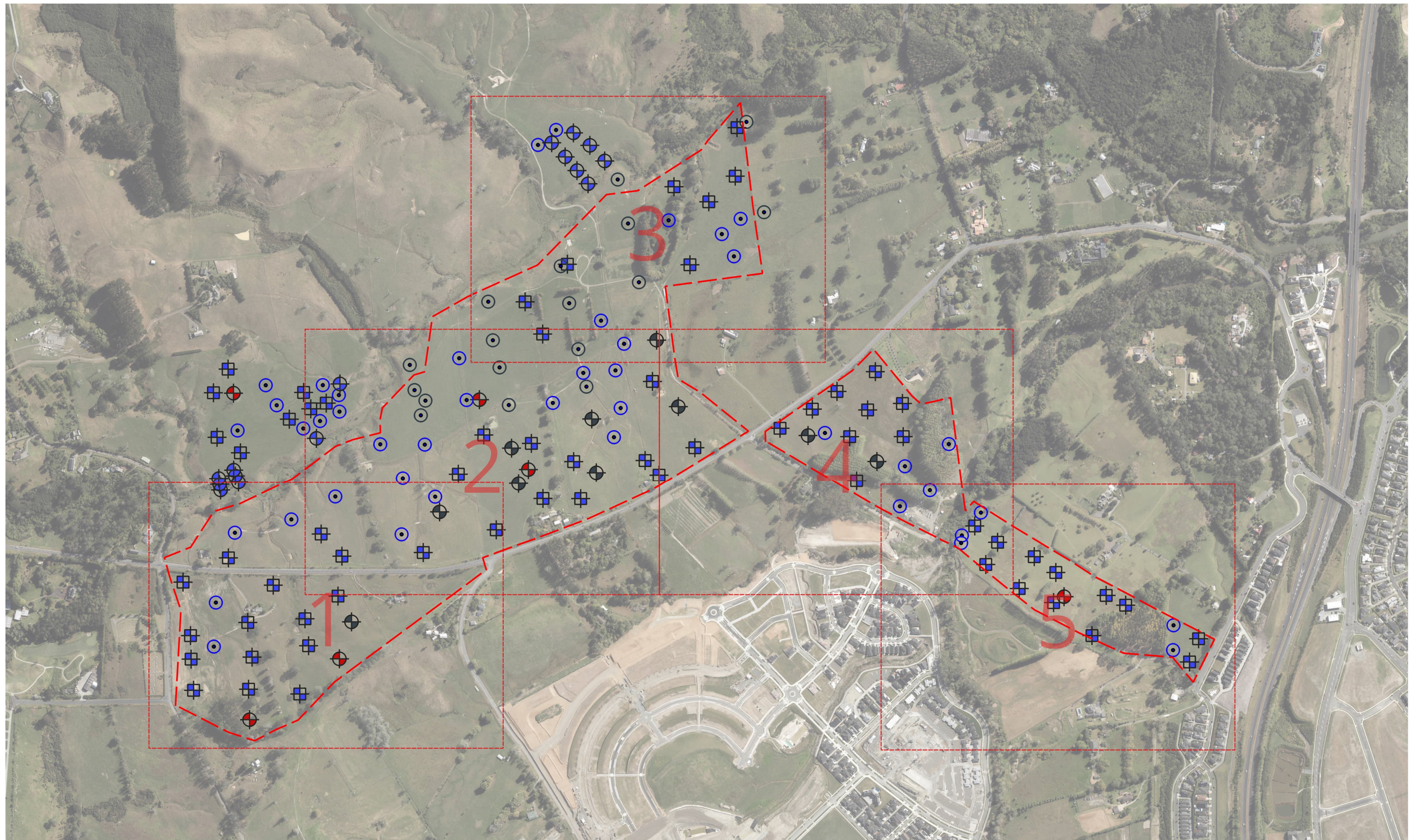
Environmental matters

Unless explicitly stated, this report does not address environmental issues such as contamination, waste management, or disposal of materials. These require separate investigation and specialist advice.

If you have any questions about the content or use of this report, please contact Gibb Ritchie Geotechnical Engineers for clarification.

APPENDIX A

Drawings



LEGEND:

- Map Grid
- Approximate Site Boundary

Existing Investigations

- CMW HAND AUGER (HA) LOCATION
- CMW TEST PIT (TP) LOCATION
- CMW MACHINE HOLE (MH) LOCATION
- CMW CPT LOCATION

Gibb Ritchie Geotechnical Engineers Investigations

- TEST PIT (TP) LOCATION
- MACHINE HOLE (MH) LOCATION
- CPT LOCATION

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— SITE BOUNDARY

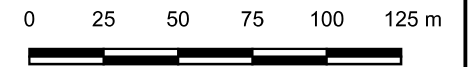
Gibb Ritchie Geotechnical Engineers Investigations

-  HAND AUGER (HA) LOCATION
-  TEST PIT (TP) LOCATION
-  MACHINE HOLE (MH) LOCATION
-  CPT LOCATION

Existing Investigations

-  CMW HAND AUGER (HA) LOCATION
-  CMW TEST PIT (TP) LOCATION
-  CMW MACHINE HOLE (MH) LOCATION
-  CMW CPT LOCATION

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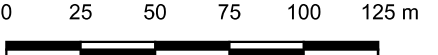
Gibb Ritchie Geotechnical Engineers Investigations

- HA HAND AUGER (HA) LOCATION
- TP TEST PIT (TP) LOCATION
- MH MACHINE HOLE (MH) LOCATION
- CPT CPT LOCATION

Existing Investigations

- CMW HA CMW HAND AUGER (HA) LOCATION
- CMW TP CMW TEST PIT (TP) LOCATION
- CMW MH CMW MACHINE HOLE (MH) LOCATION
- CMW CPT CMW CPT LOCATION

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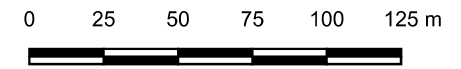
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-  MACHINE HOLE (MH) LOCATION
-  CPT LOCATION

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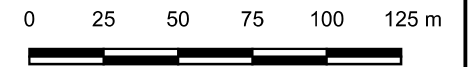
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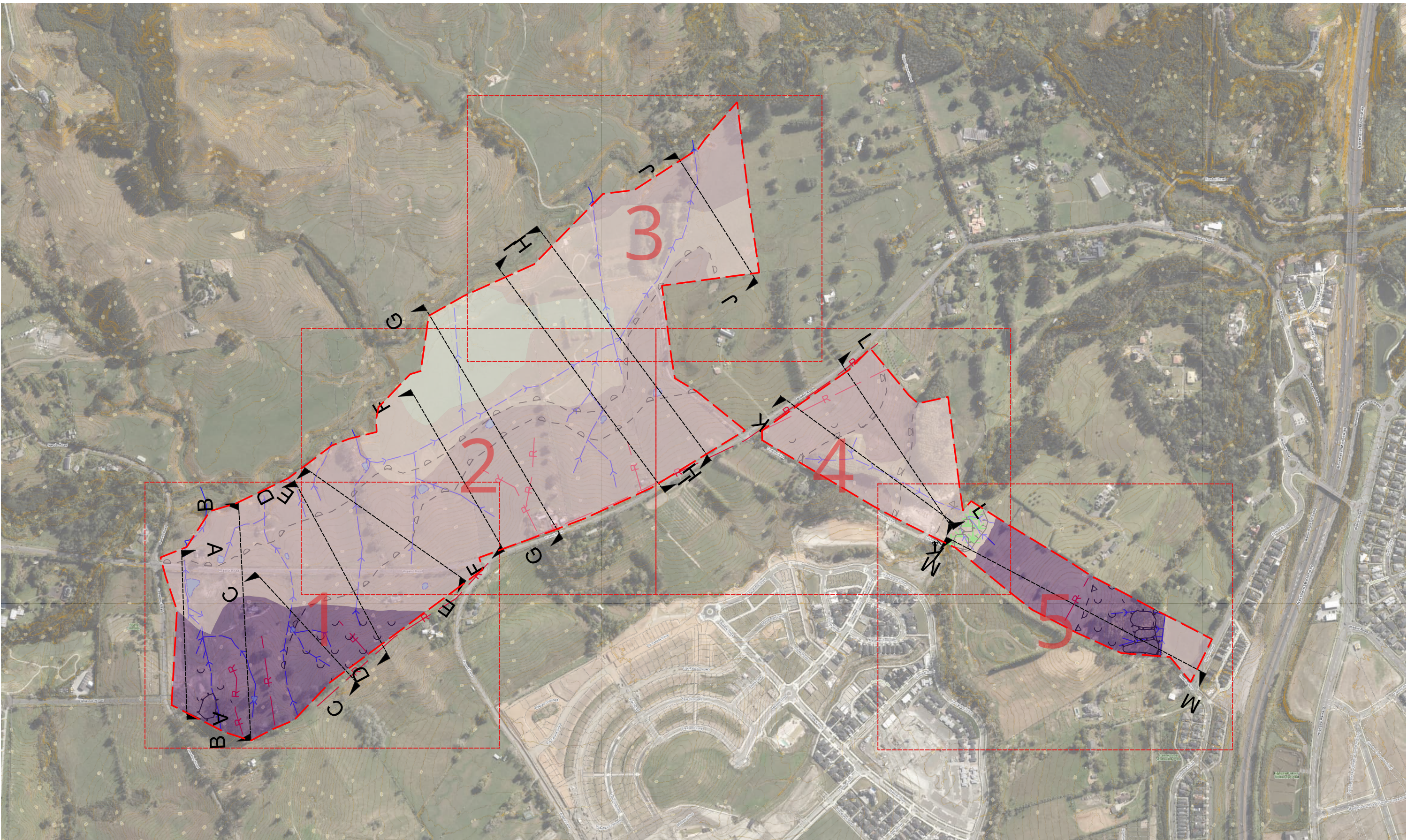
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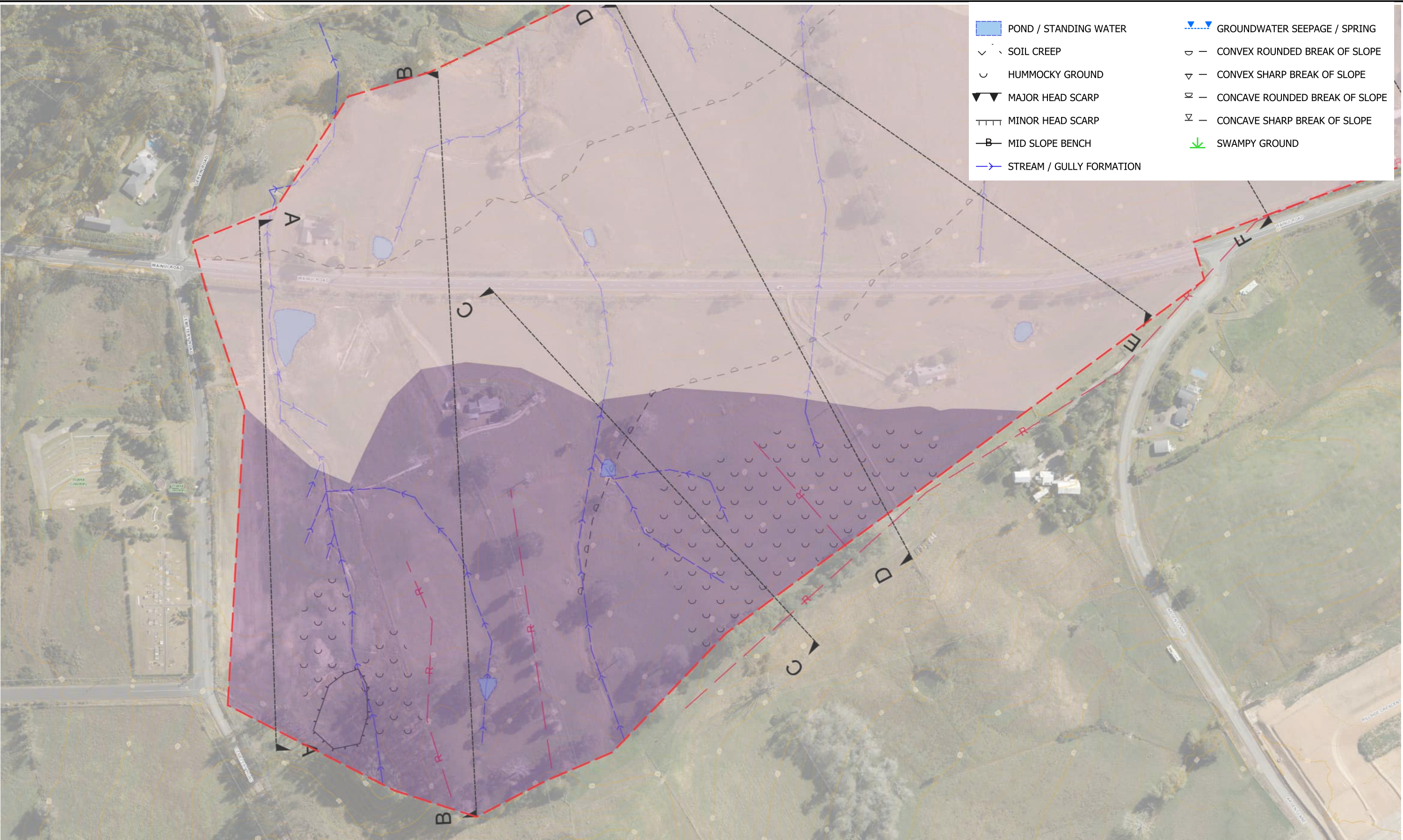
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- Map Grid
- Approximate Site Boundary

- Intepreted Geology
- Alluvium Soft (Q1a, Holocene River Deposits)
 - Alluvium Firm (Q1a, Holocene River Deposits)
 - Alluvium Stiff (IQa, Middle to Late Plesitocene River and Hill Deposits)
 - Hukerenui Mudstone (KKh, Northland Allochthon)

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POND / STANDING WATER

SOIL CREEP

HUMMOCKY GROUND

MAJOR HEAD SCARP

MINOR HEAD SCARP

MID SLOPE BENCH

STREAM / GULLY FORMATION

GROUNDWATER SEEPAGE / SPRING

CONVEX ROUNDED BREAK OF SLOPE

CONVEX SHARP BREAK OF SLOPE

CONCAVE ROUNDED BREAK OF SLOPE

CONCAVE SHARP BREAK OF SLOPE

SWAMPY GROUND

LEGEND:

Map Grid

Approximate Site Boundary

Cross Section Locations

Intepreted Geology

Alluvium Soft (Q1a, Holocene River Deposits)

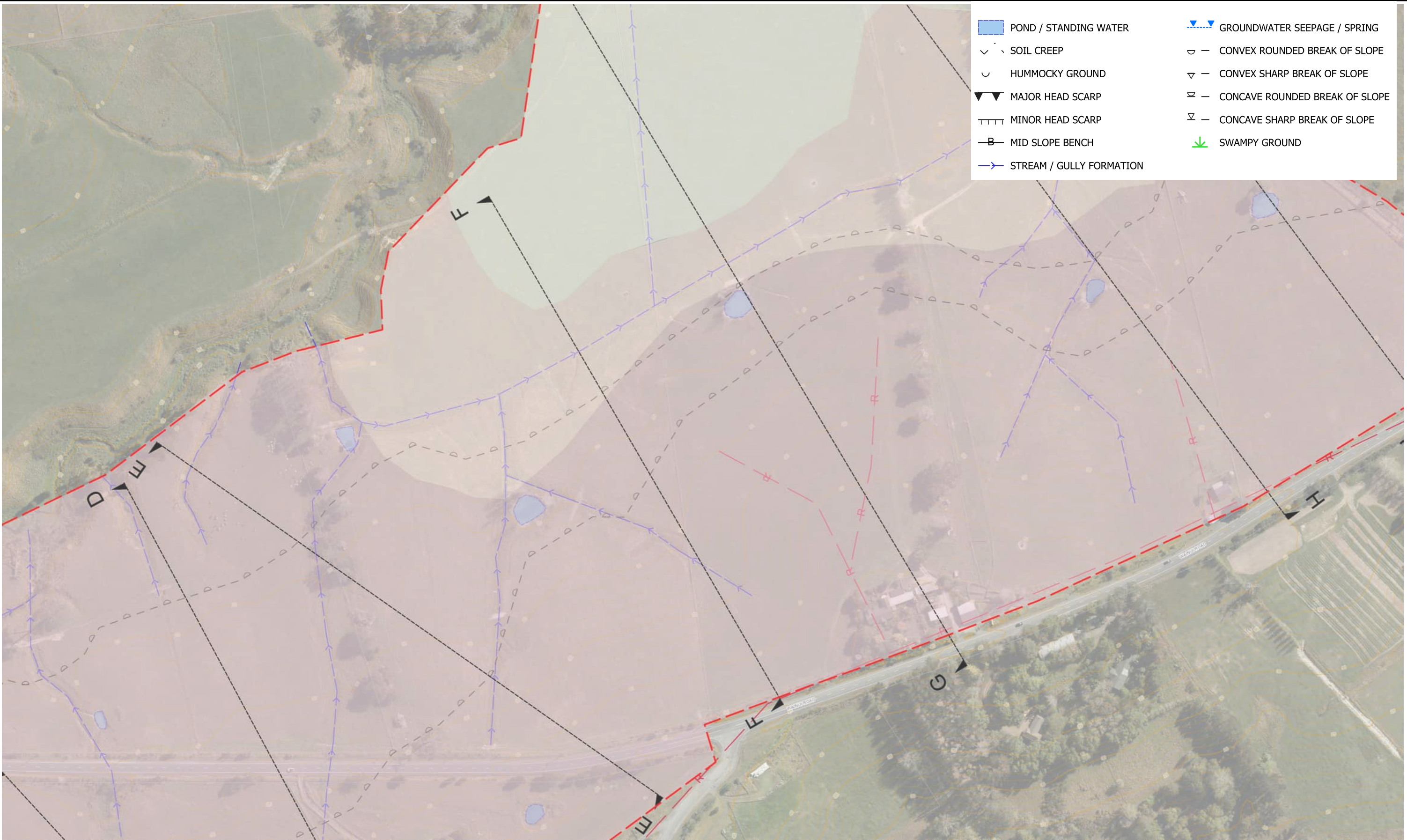
Alluvium Firm (Q1a, Holocene River Deposits)

Alluvium Stiff (IQa, Middle to Late Plesitocene River and Hill Deposits)

Hukerenui Mudstone (KKh, Northland Allochthon)

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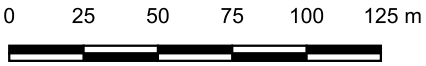
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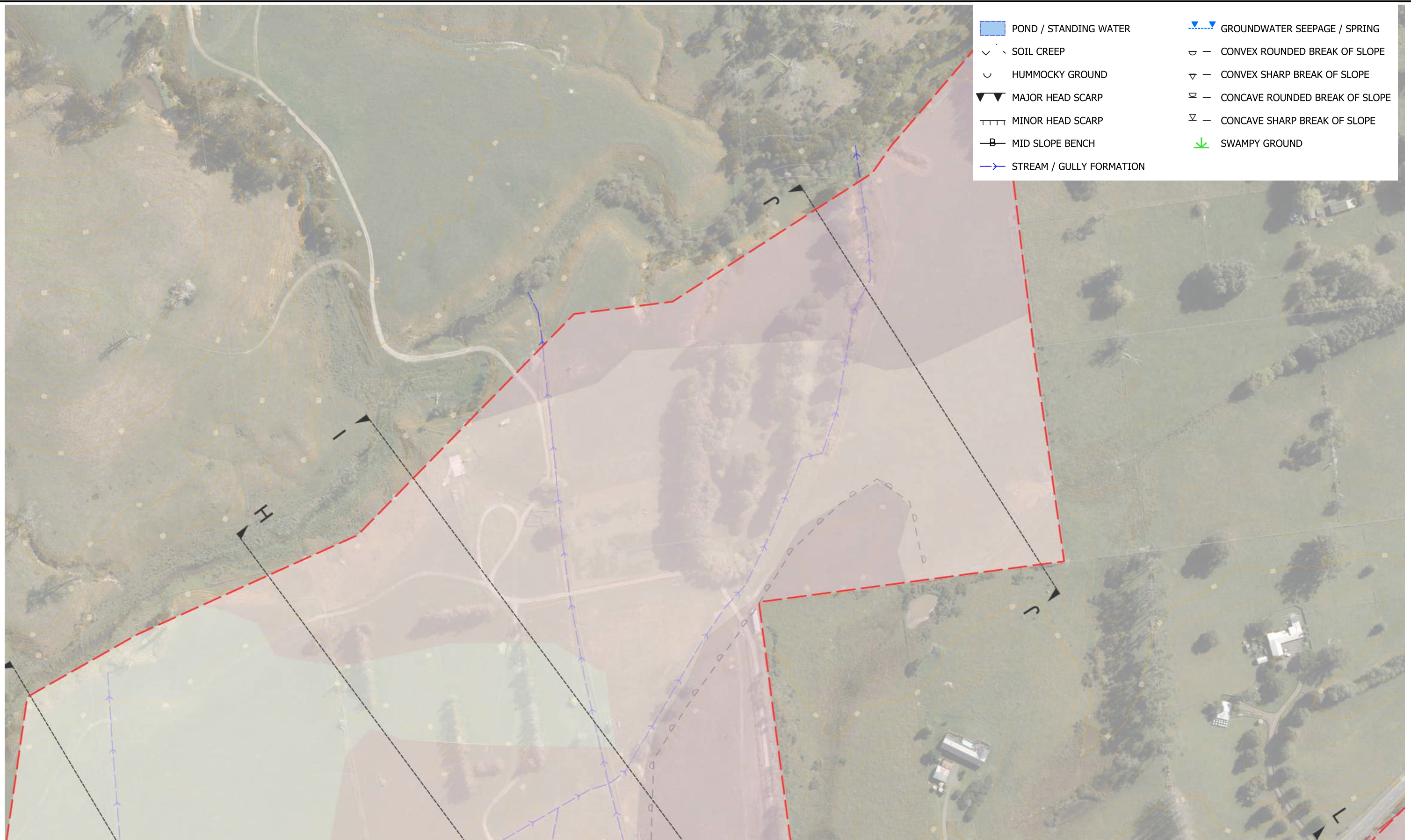
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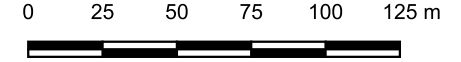


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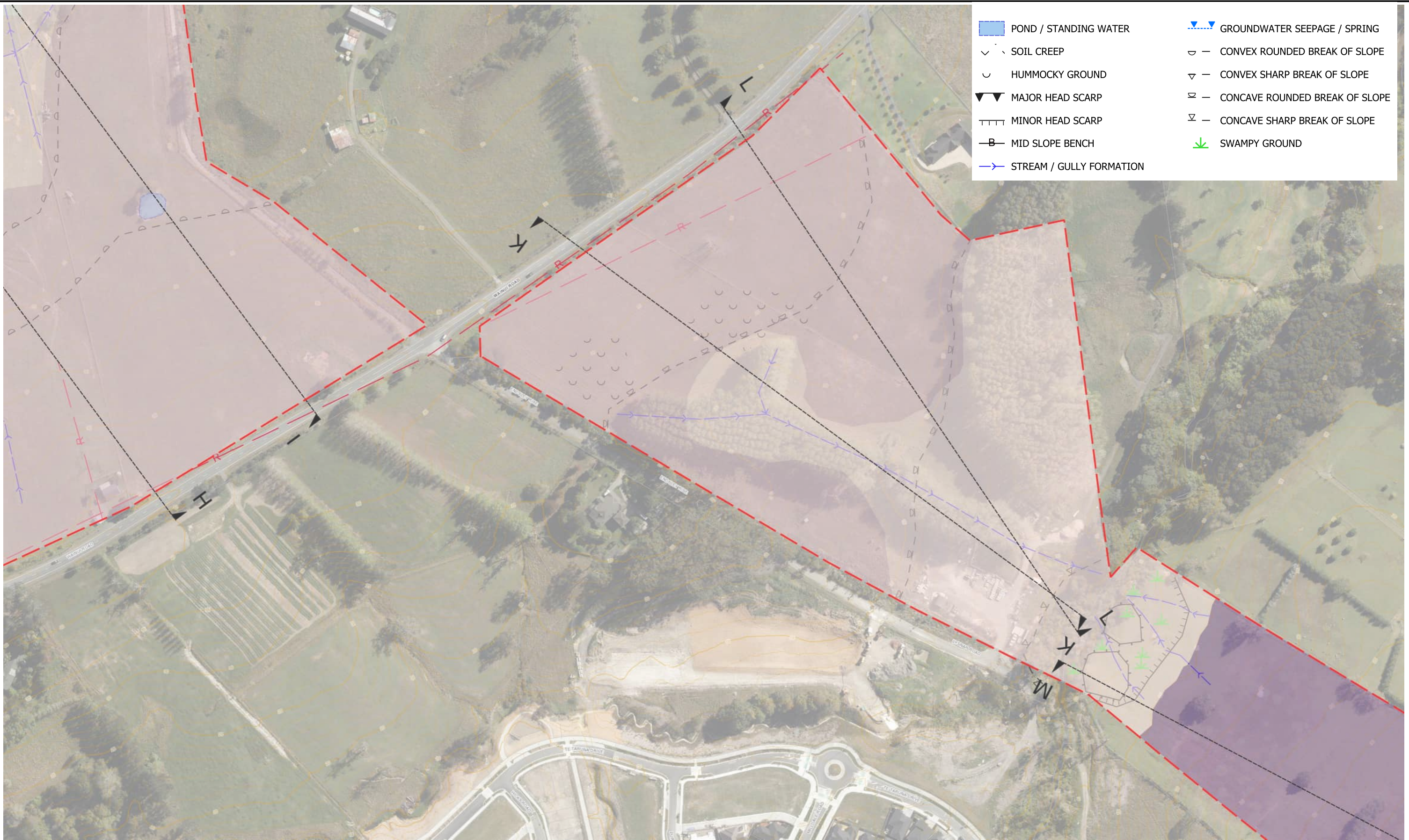
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- Cross Section Locations

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- Cross Section Locations

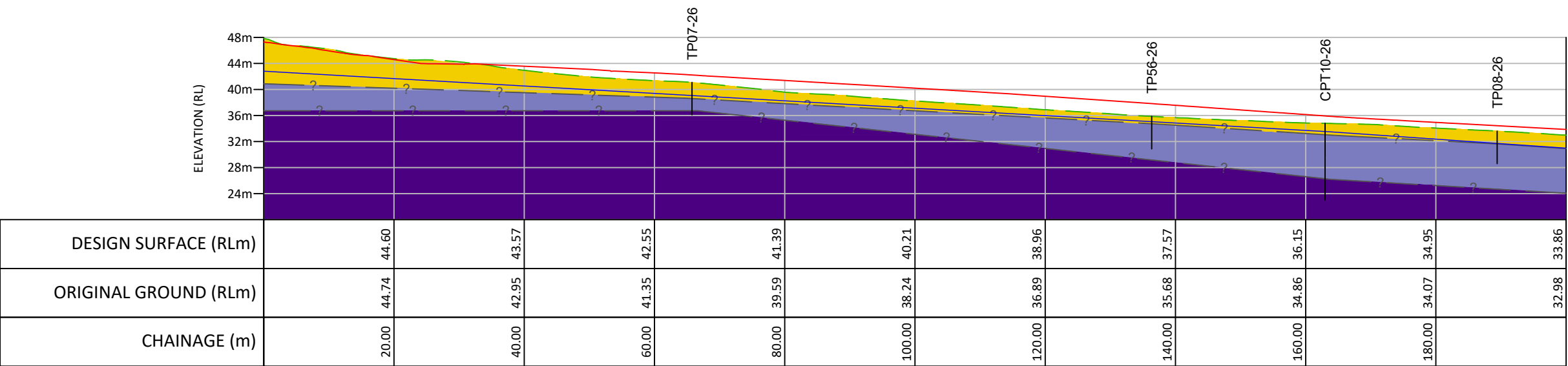
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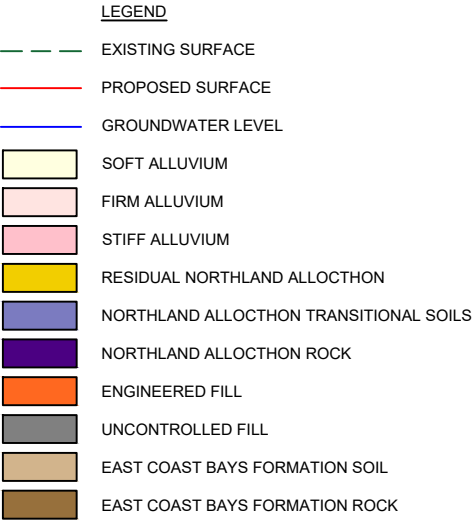
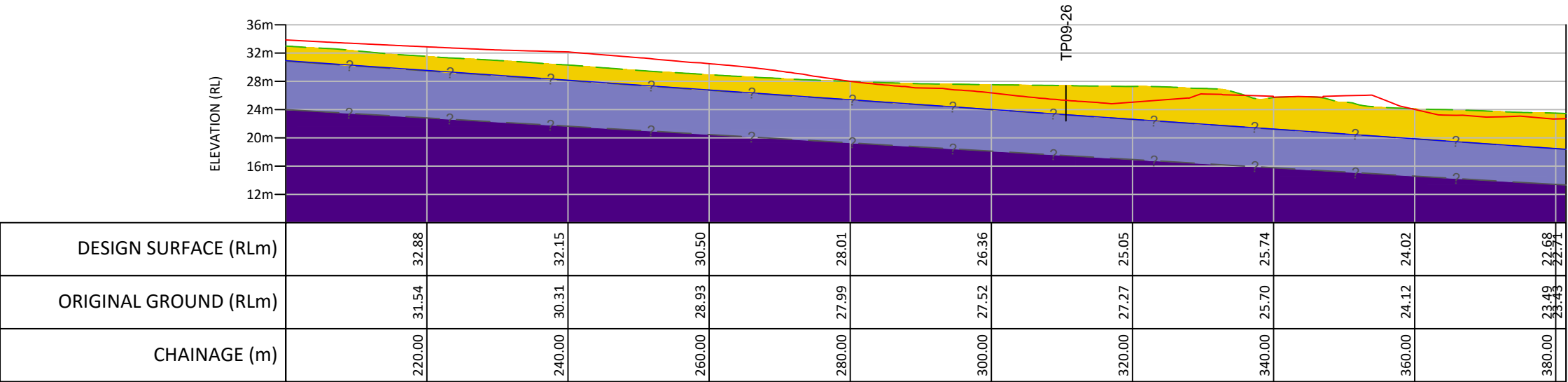
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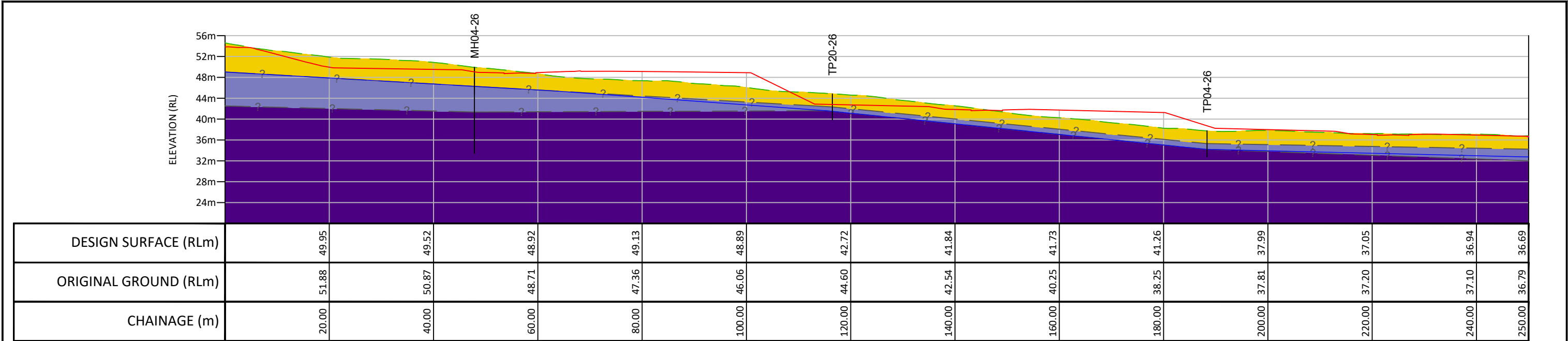
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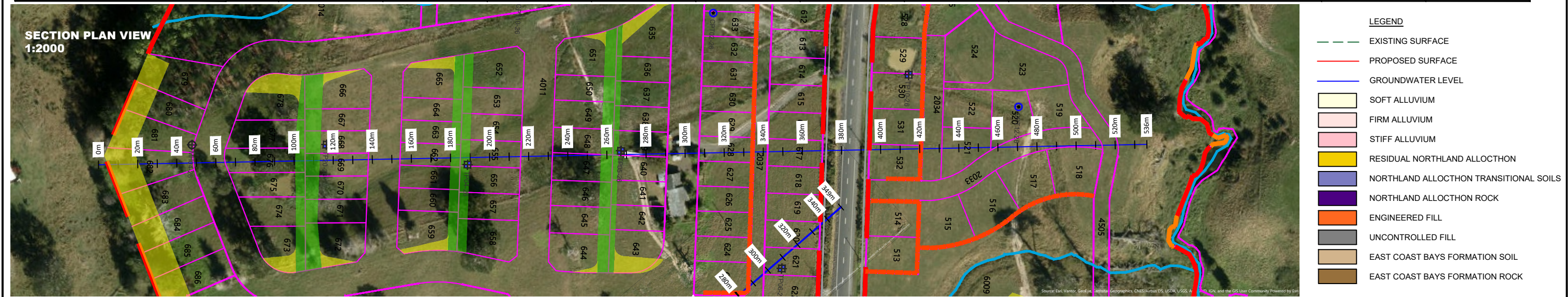
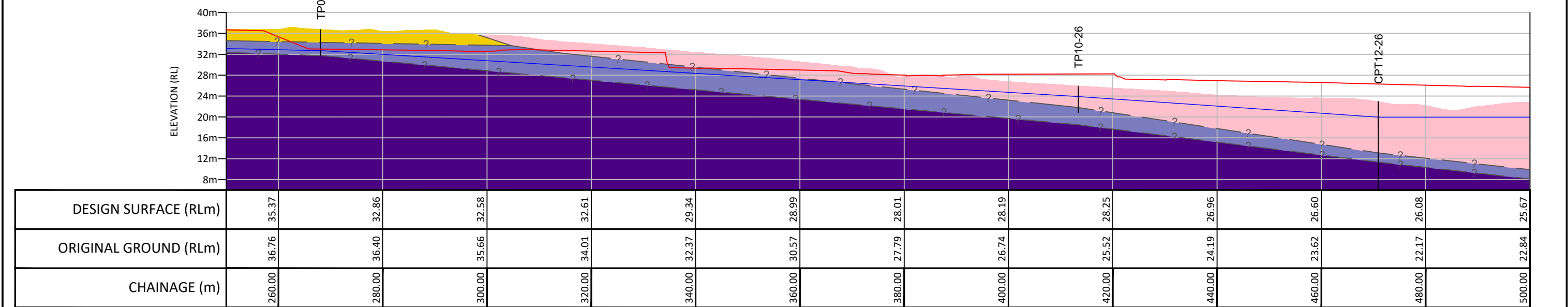
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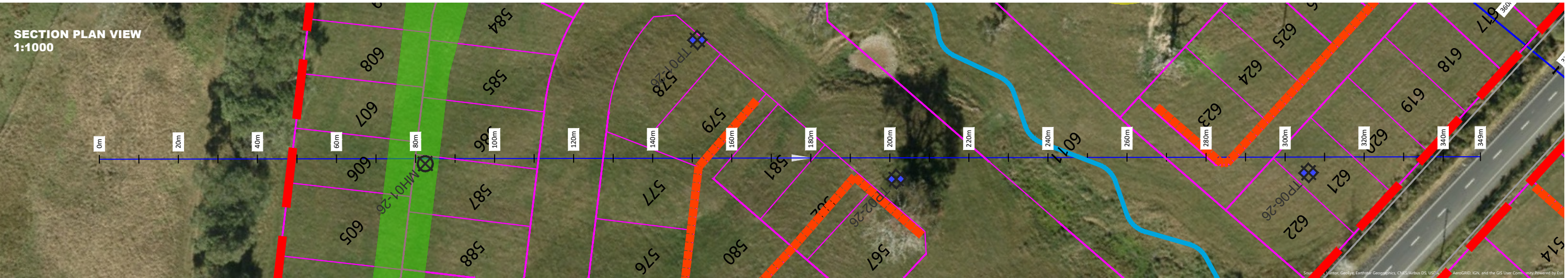
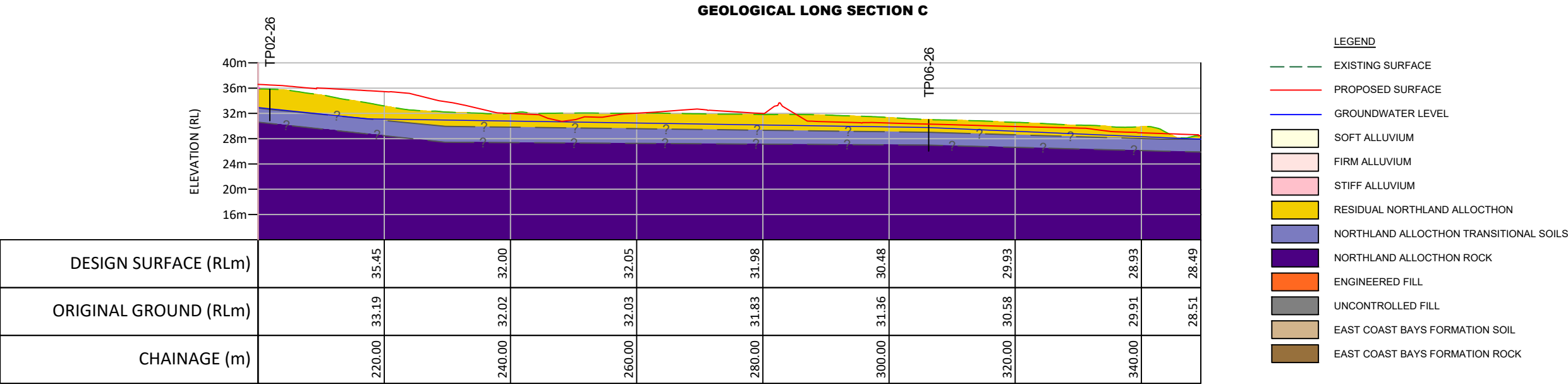
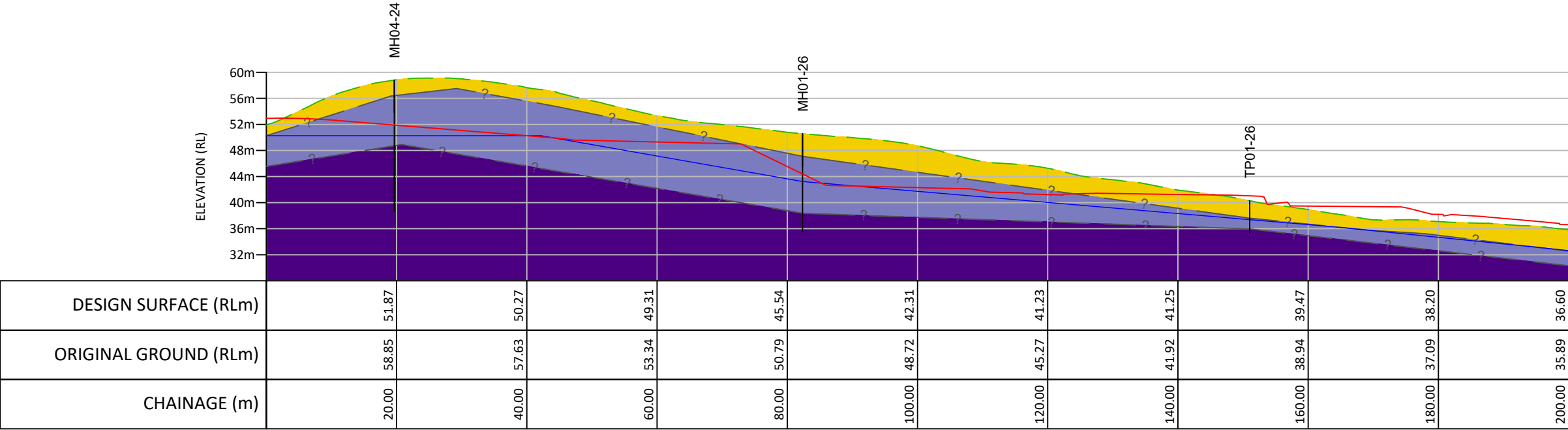
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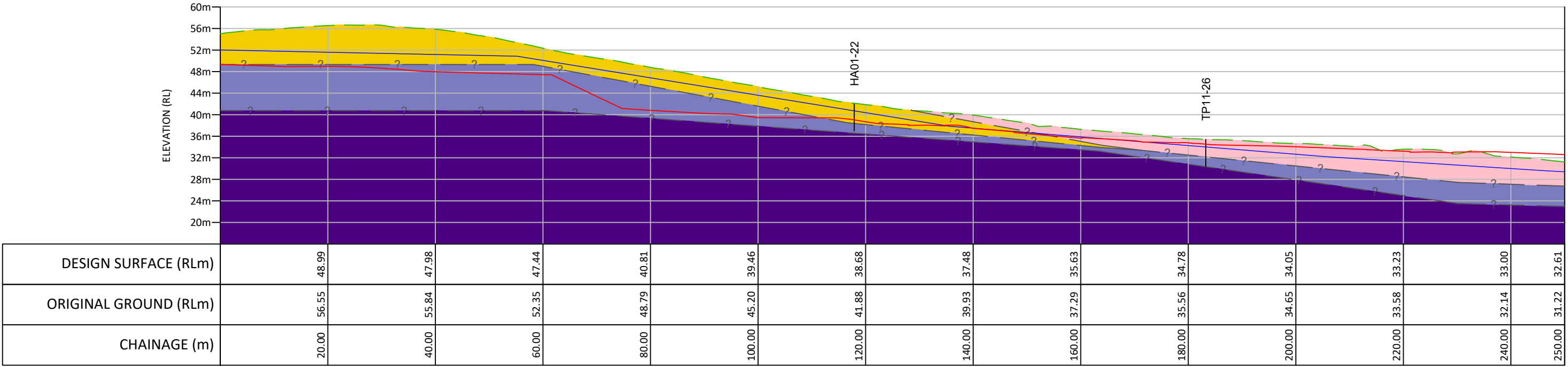
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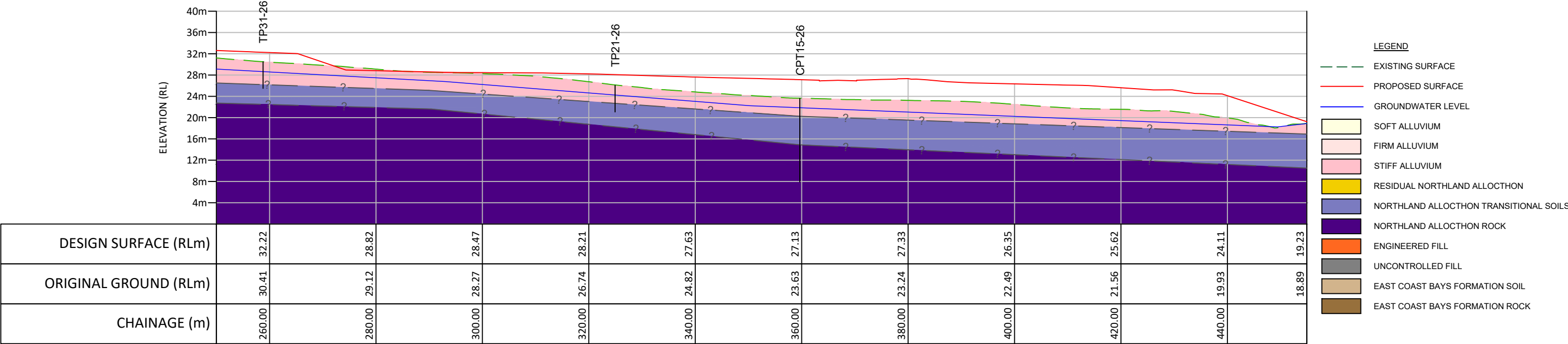
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0	03/06/26	FOR CLIENT REVIEW	CR
1	23/07/26	FINAL	CR
REV	DATE	DESCRIPTION	BY



CLIENT:	FULTON HOGAN LAND DEVELOPMENT		DRAWN:	CR	PROJECT:	AK26-027
PROJECT:	MILLDALE NORTH FAST TRACK MILLDALE		CHECKED:	SG	FIGURE:	03
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TITLE:	GEOLOGICAL LONG SECTION C		DATE:	03/06/26	SHEET:	A3



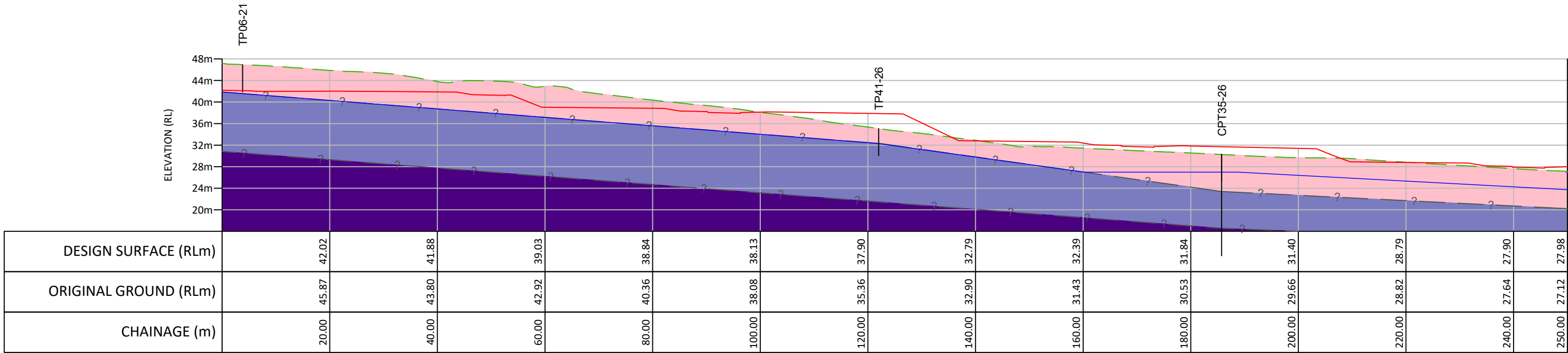
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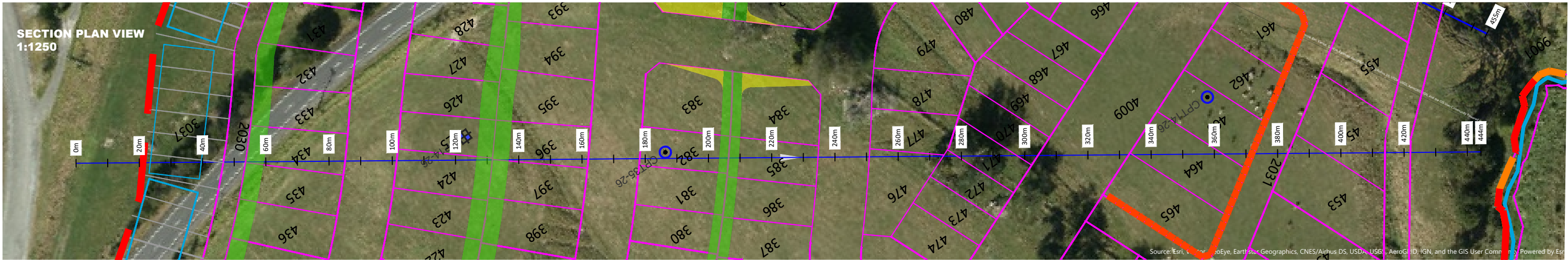
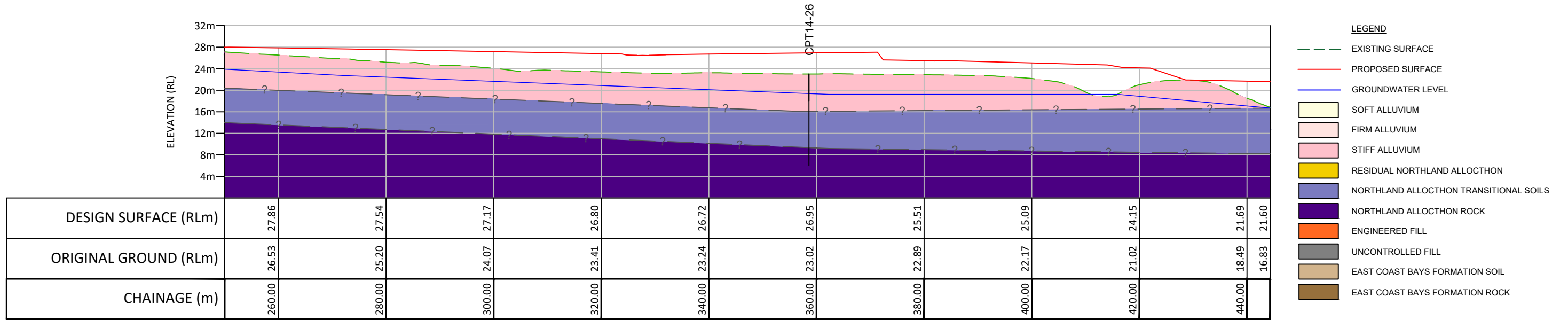
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1	23/07/26	FINAL	CR
REV	DATE	DESCRIPTION	BY



CLIENT:	FULTON HOGAN LAND DEVELOPMENT		DRAWN:	CR	PROJECT:	AK26-027
PROJECT:	MILLDALE NORTH FAST TRACK MILLDALE		CHECKED:	SG	FIGURE:	03
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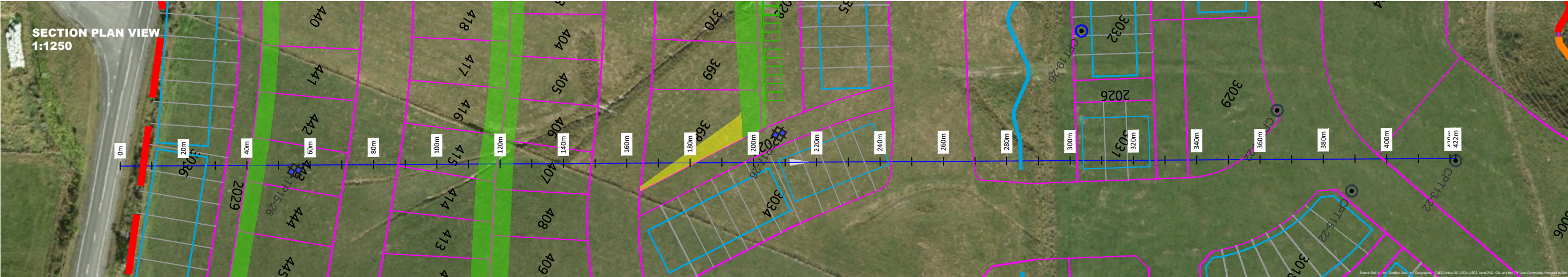
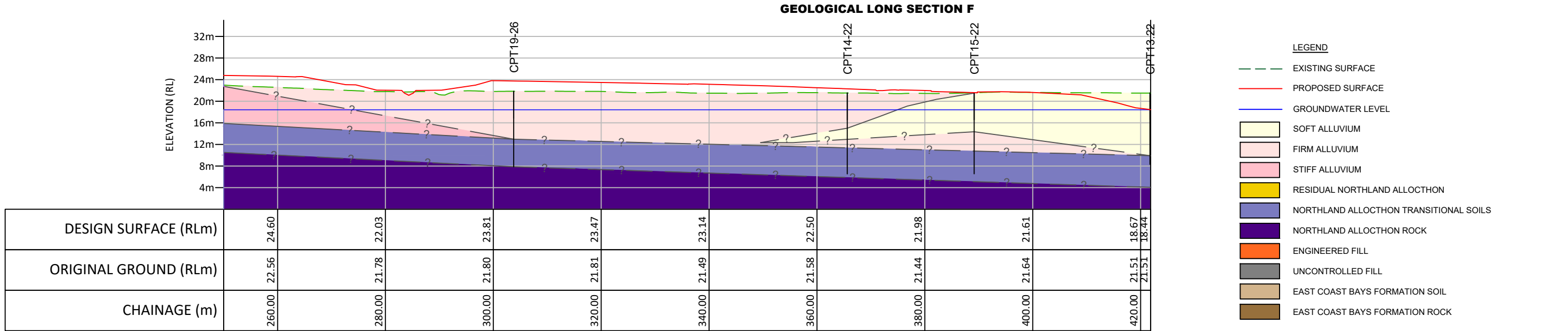
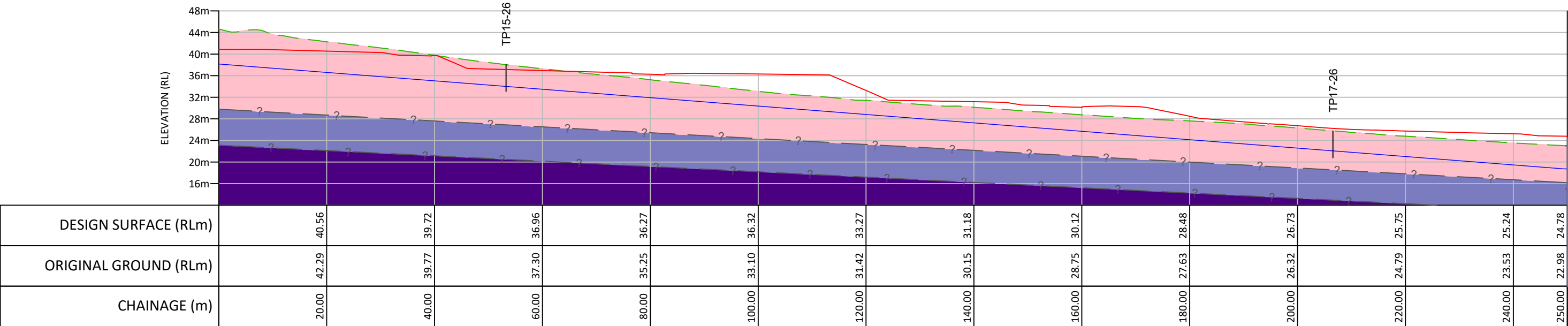
GEOLOGICAL LONG SECTION E



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REV	DATE	DESCRIPTION	BY



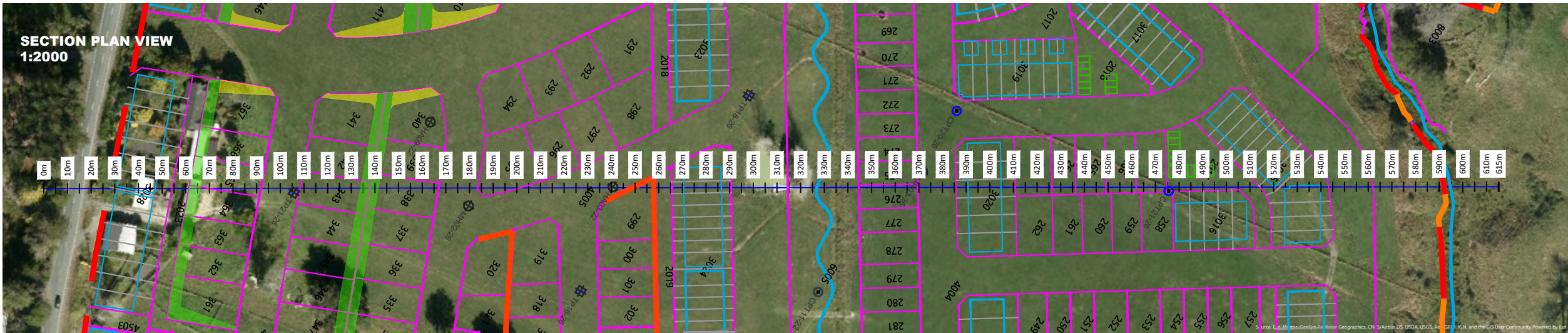
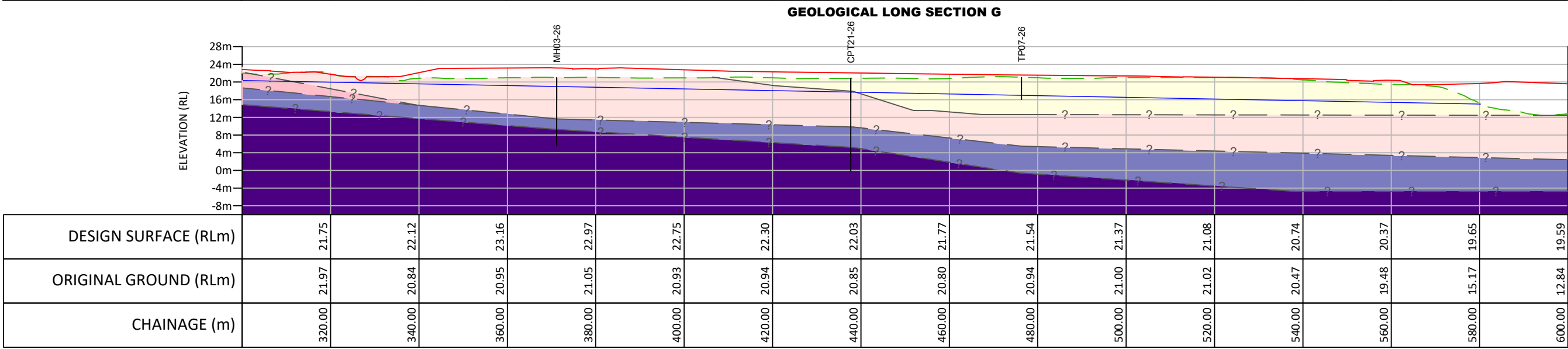
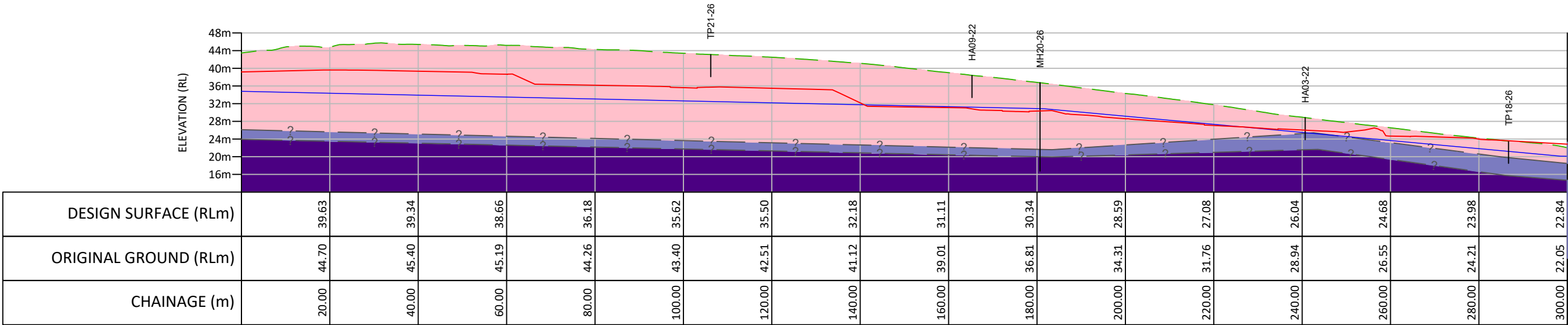
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PROJECT:	MILLDALE NORTH FAST TRACK MILLDALE		CHECKED:	SG	FIGURE:	03
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1	23/07/26	FINAL	CR
REV	DATE	DESCRIPTION	BY



CLIENT:	FULTON HOGAN LAND DEVELOPMENT		DRAWN:	CR	PROJECT:	AK26-027
PROJECT:	MILLDALE NORTH FAST TRACK MILLDALE		CHECKED:	SG	FIGURE:	03
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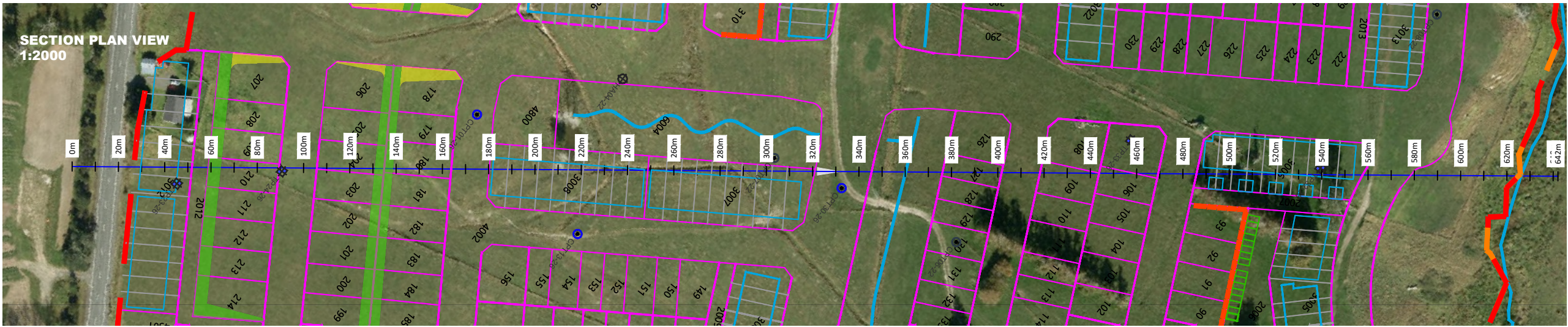
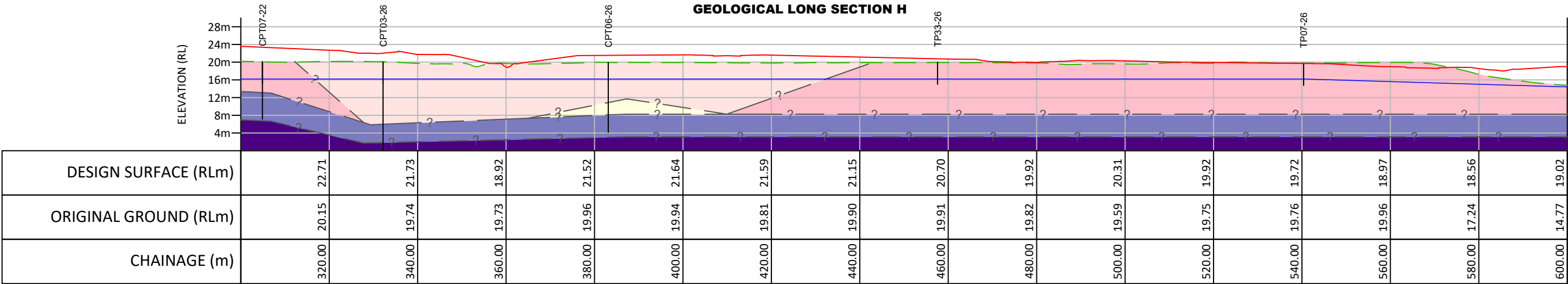
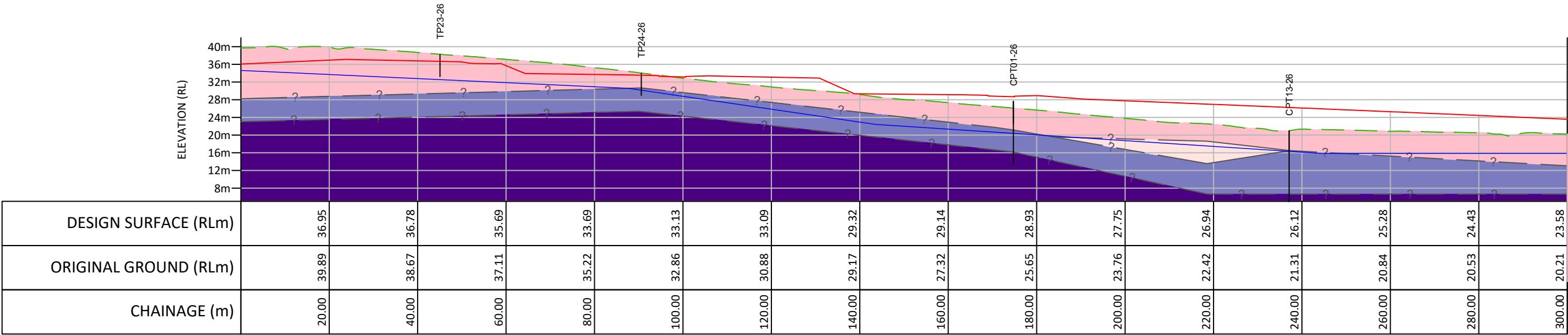


- LEGEND**
- EXISTING SURFACE
 - PROPOSED SURFACE
 - GROUNDWATER LEVEL
 - SOFT ALLUVIUM
 - FIRM ALLUVIUM
 - STIFF ALLUVIUM
 - RESIDUAL NORTHLAND ALLOCTHON
 - NORTHLAND ALLOCTHON TRANSITIONAL SOILS
 - NORTHLAND ALLOCTHON ROCK
 - ENGINEERED FILL
 - UNCONTROLLED FILL
 - EAST COAST BAYS FORMATION SOIL
 - EAST COAST BAYS FORMATION ROCK

A	15/04/26	FOR INTERNAL REVIEW	CR
0	03/06/26	FOR CLIENT REVIEW	CR
1	23/07/26	FINAL	CR
REV	DATE	DESCRIPTION	BY



CLIENT:	FULTON HOGAN LAND DEVELOPMENT		DRAWN:	CR	PROJECT:	AK26-027
PROJECT:	MILLDALE NORTH FAST TRACK MILLDALE		CHECKED:	SG	FIGURE:	03
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TITLE:	GEOLOGICAL LONG SECTION G		DATE:	03/06/26	SHEET:	A3



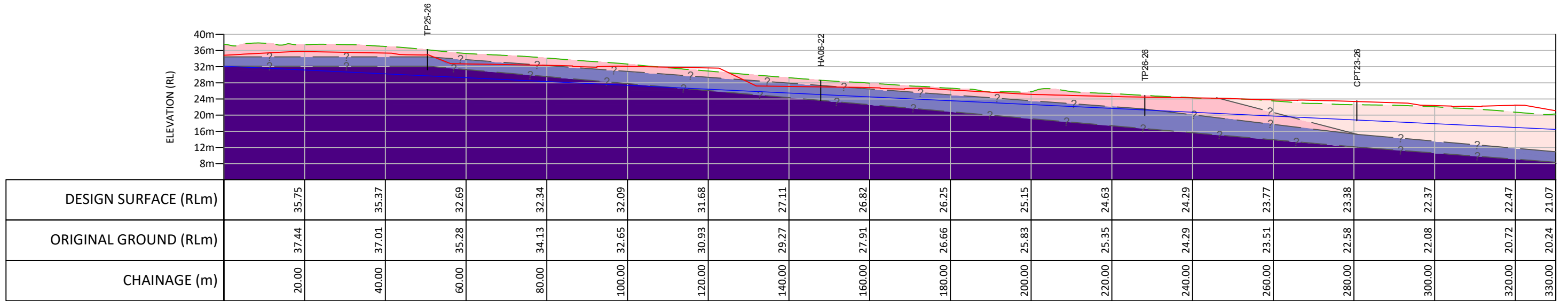
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	PROPOSED SURFACE
	GROUNDWATER LEVEL
	SOFT ALLUVIUM
	FIRM ALLUVIUM
	STIFF ALLUVIUM
	RESIDUAL NORTHLAND ALLOCTION
	NORTHLAND ALLOCTION TRANSITIONAL
	NORTHLAND ALLOCTION ROCK
	ENGINEERED FILL
	UNCONTROLLED FILL
	EAST COAST BAYS FORMATION SOIL
	EAST COAST BAYS FORMATION ROCK

A	15/04/26	FOR INTERNAL REVIEW	CR
0	03/06/26	FOR CLIENT REVIEW	CR
1	23/07/26	FINAL	CR
REV	DATE	DESCRIPTION	BY

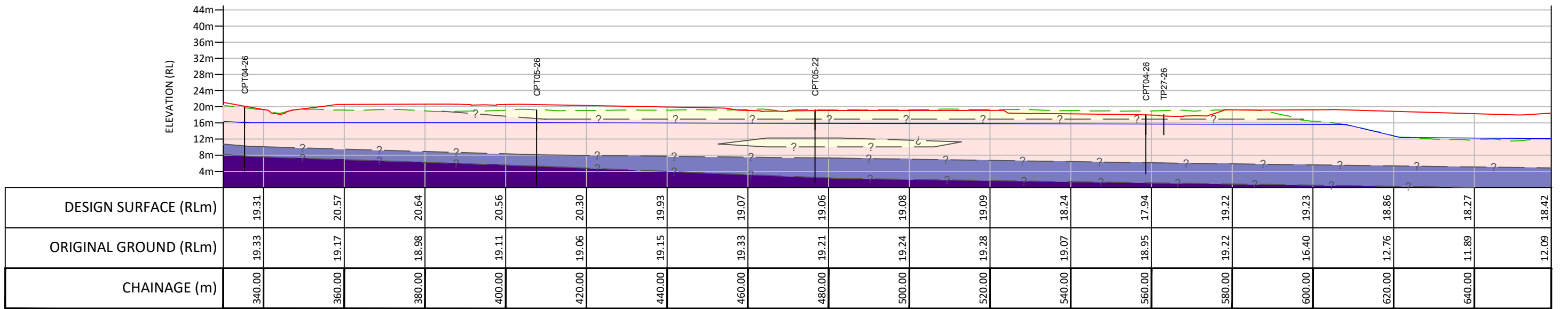


CLIENT:	FULTON HOGAN LAND DEVELOPMENT	
PROJECT:	MILLDALE NORTH FAST TRACK MILLDALE	
TITLE:	GEOLOGICAL LONG SECTION H	

DRAWN:	CR	PROJECT:	AK26-027
CHECKED:	SG	FIGURE:	03
REVISION:	0	SCALE:	1:1000
DATE:	03/06/26	SHEET:	A3



GEOLOGICAL LONG SECTION I



- LEGEND**
- EXISTING SURFACE
 - PROPOSED SURFACE
 - GROUNDWATER LEVEL
 - SOFT ALLUVIUM
 - FIRM ALLUVIUM
 - STIFF ALLUVIUM
 - RESIDUAL NORTHLAND ALLOCTHON
 - NORTHLAND ALLOCTHON TRANSITION
 - NORTHLAND ALLOCTHON ROCK
 - ENGINEERED FILL
 - UNCONTROLLED FILL
 - EAST COAST BAYS FORMATION SOIL
 - EAST COAST BAYS FORMATION ROCK

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0	03/06/26	FOR CLIENT REVIEW	CR
1	23/07/26	FINAL	CR
REV	DATE	DESCRIPTION	BY

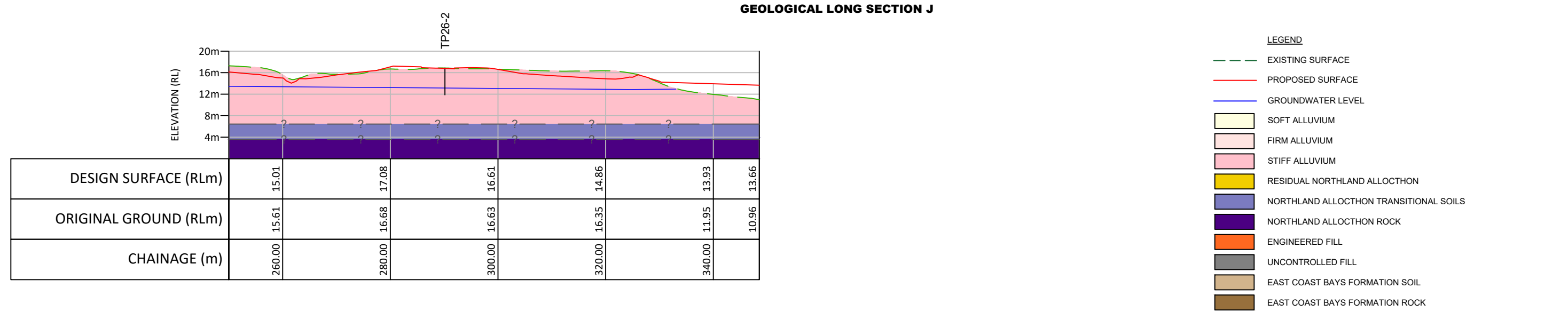
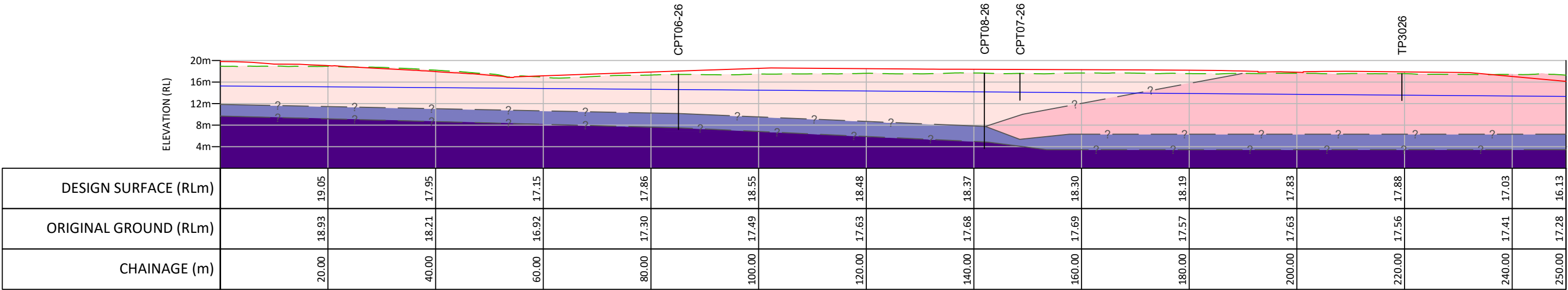


CLIENT: **FULTON HOGAN LAND DEVELOPMENT**

PROJECT: **MILLDALE NORTH FAST TRACK
MILLDALE**

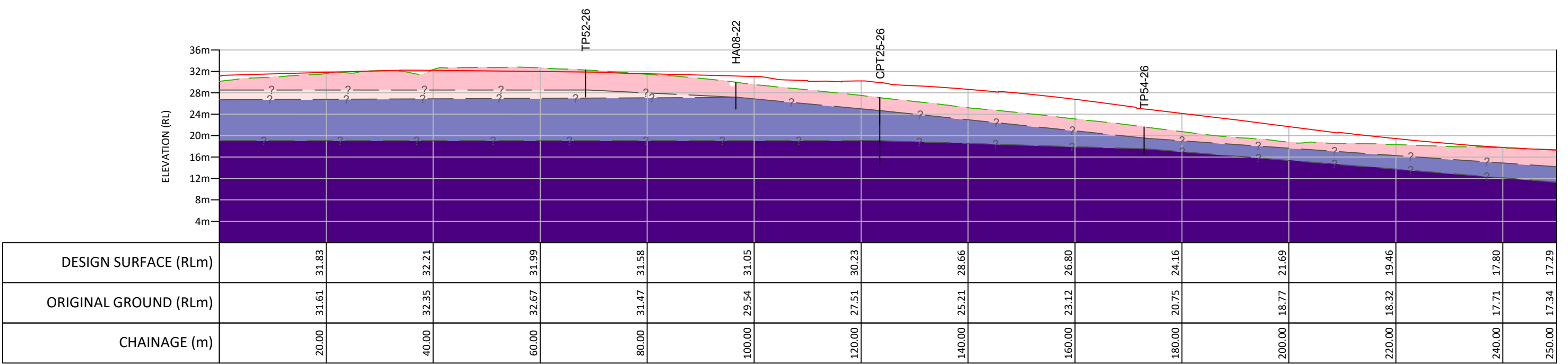
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CHECKED:	SG	FIGURE:	03
REVISION:	0	SCALE:	1:1000
DATE:	03/06/26	SHEET:	A3

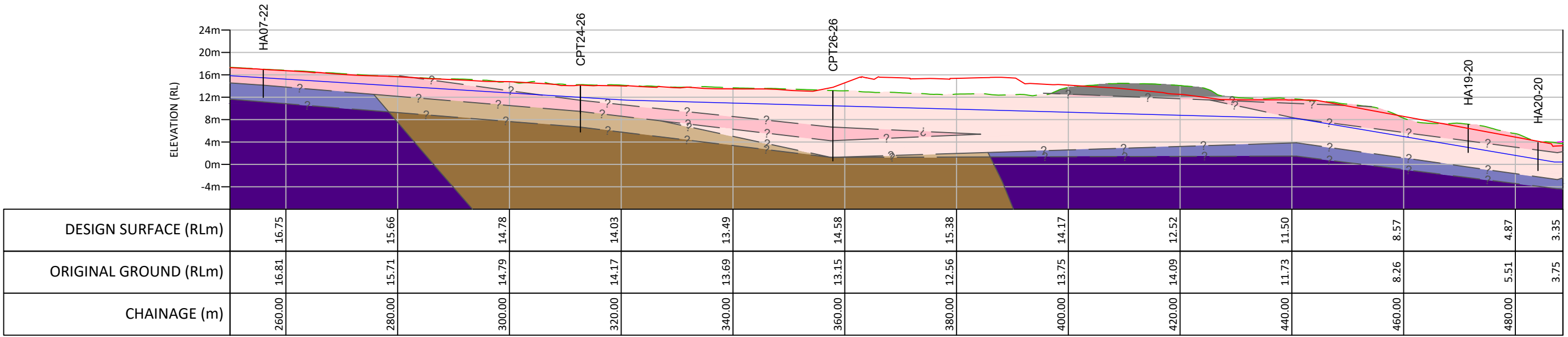


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REV	DATE	DESCRIPTION	BY

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	PROJECT:	MILLDALE NORTH FAST TRACK MILLDALE		CHECKED:	SG	FIGURE:	03
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GEOLOGICAL LONG SECTION K



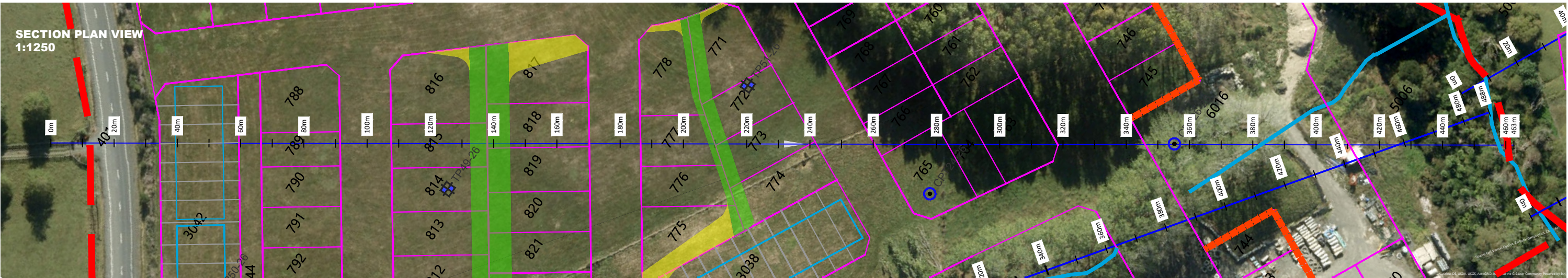
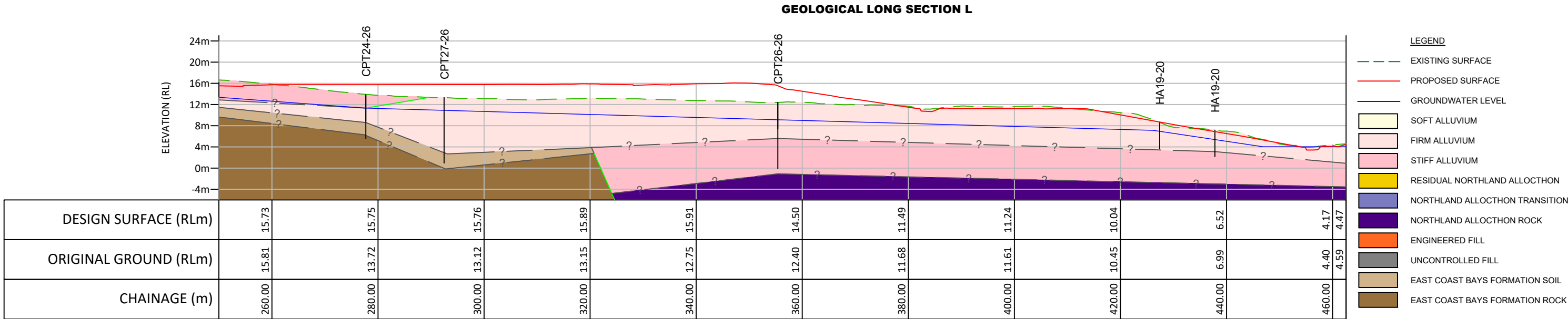
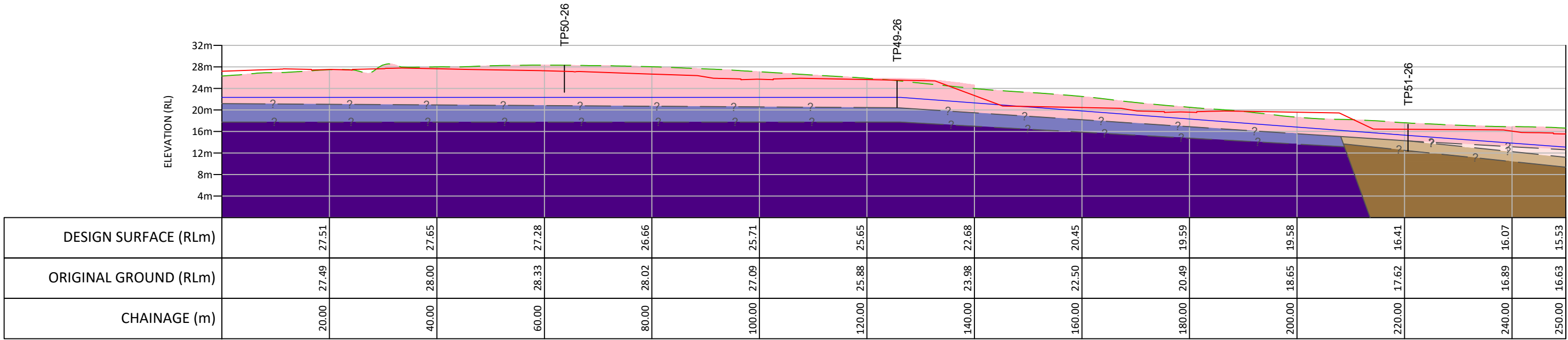
LEGEND	
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	PROPOSED SURFACE
	GROUNDWATER LEVEL
	SOFT ALLUVIUM
	FIRM ALLUVIUM
	STIFF ALLUVIUM
	RESIDUAL NORTHLAND ALLOCTION
	NORTHLAND ALLOCTION TRANSITIONAL SOILS
	NORTHLAND ALLOCTION ROCK
	ENGINEERED FILL
	UNCONTROLLED FILL
	EAST COAST BAYS FORMATION SOIL
	EAST COAST BAYS FORMATION ROCK

A	15/04/26	FOR INTERNAL REVIEW	CR
0	03/06/26	FOR CLIENT REVIEW	CR
1	23/07/26	FINAL	CR
REV	DATE	DESCRIPTION	BY



CLIENT:	FULTON HOGAN LAND DEVELOPMENT
PROJECT:	MILLDALE NORTH FAST TRACK MILLDALE
TITLE:	GEOLOGICAL LONG SECTION K

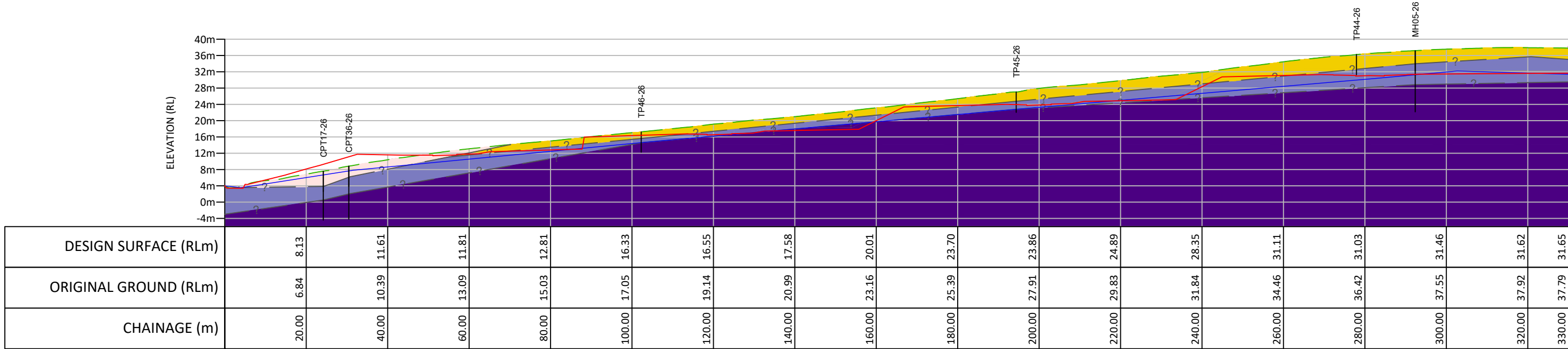
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CHECKED:	SG	FIGURE:	03
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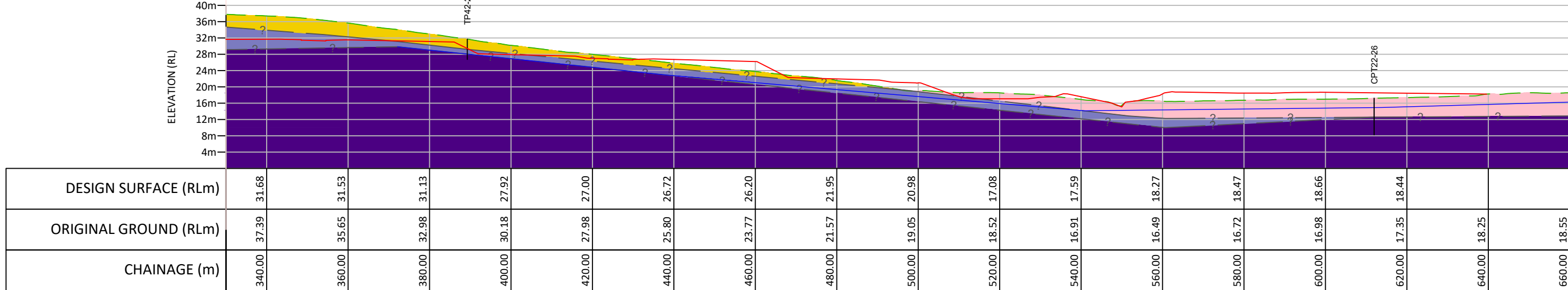
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0	03/06/26	FOR CLIENT REVIEW	CR
1	23/07/26	FINAL	CR
REV	DATE	DESCRIPTION	BY



CLIENT:	FULTON HOGAN LAND DEVELOPMENT		DRAWN:	CR	PROJECT:	AK26-027
PROJECT:	MILLDALE NORTH FAST TRACK MILLDALE		CHECKED:	SG	FIGURE:	03
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GEOLOGICAL LONG SECTION M

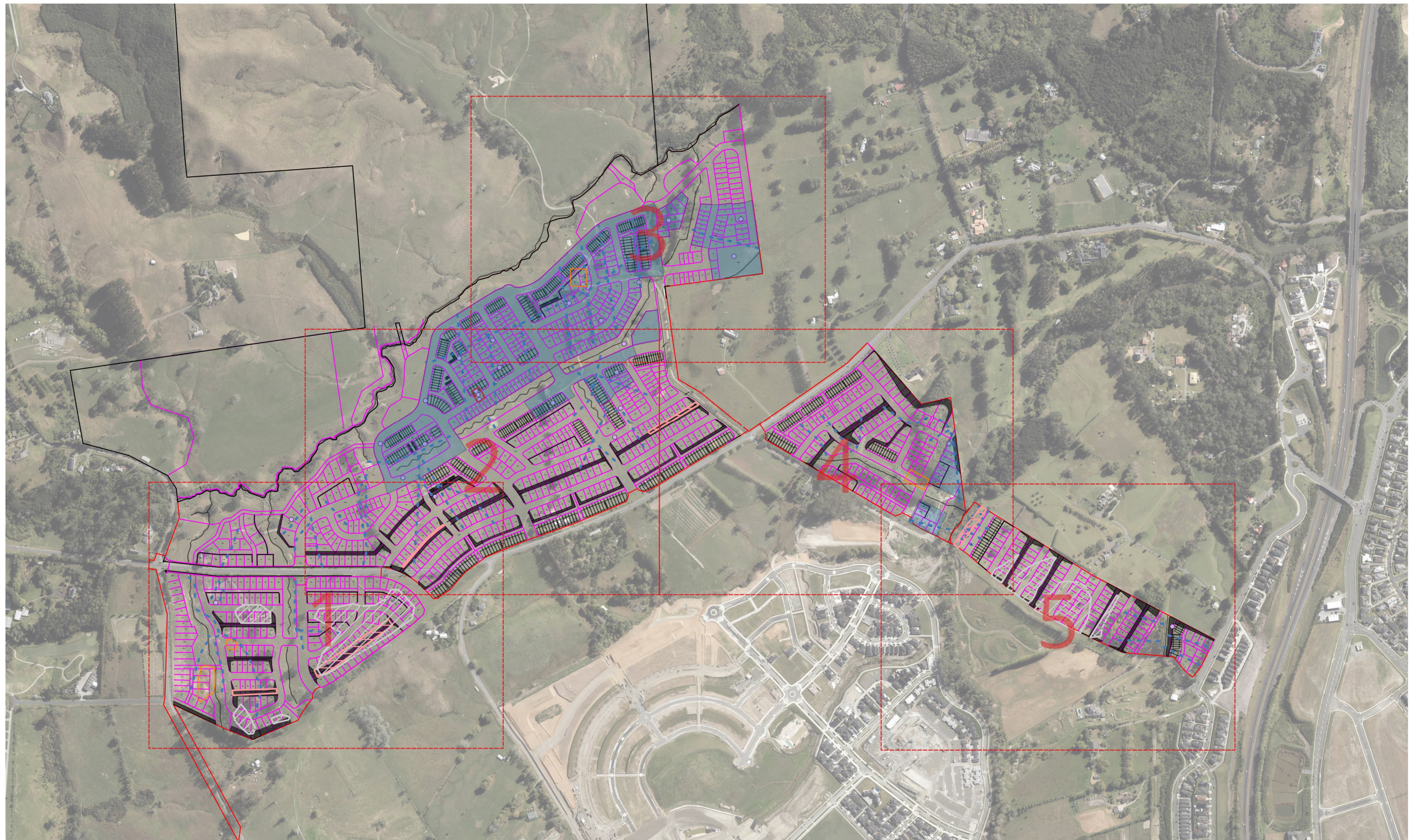


- LEGEND
- EXISTING SURFACE
 - PROPOSED SURFACE
 - GROUNDWATER LEVEL
 - SOFT ALLUVIUM
 - FIRM ALLUVIUM
 - STIFF ALLUVIUM
 - RESIDUAL NORTHLAND ALLOCTION
 - NORTHLAND ALLOCTION TRANSITIONAL SOILS
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 - EAST COAST BAYS FORMATION ROCK

A	15/04/26	FOR INTERNAL REVIEW	CR
0	03/06/26	FOR CLIENT REVIEW	CR
1	23/07/26	FINAL	CR
REV	DATE	DESCRIPTION	BY



CLIENT:	FULTON HOGAN LAND DEVELOPMENT		DRAWN:	CR	PROJECT:	AK26-027
PROJECT:	MILLDALE NORTH FAST TRACK MILLDALE		CHECKED:	SG	FIGURE:	03
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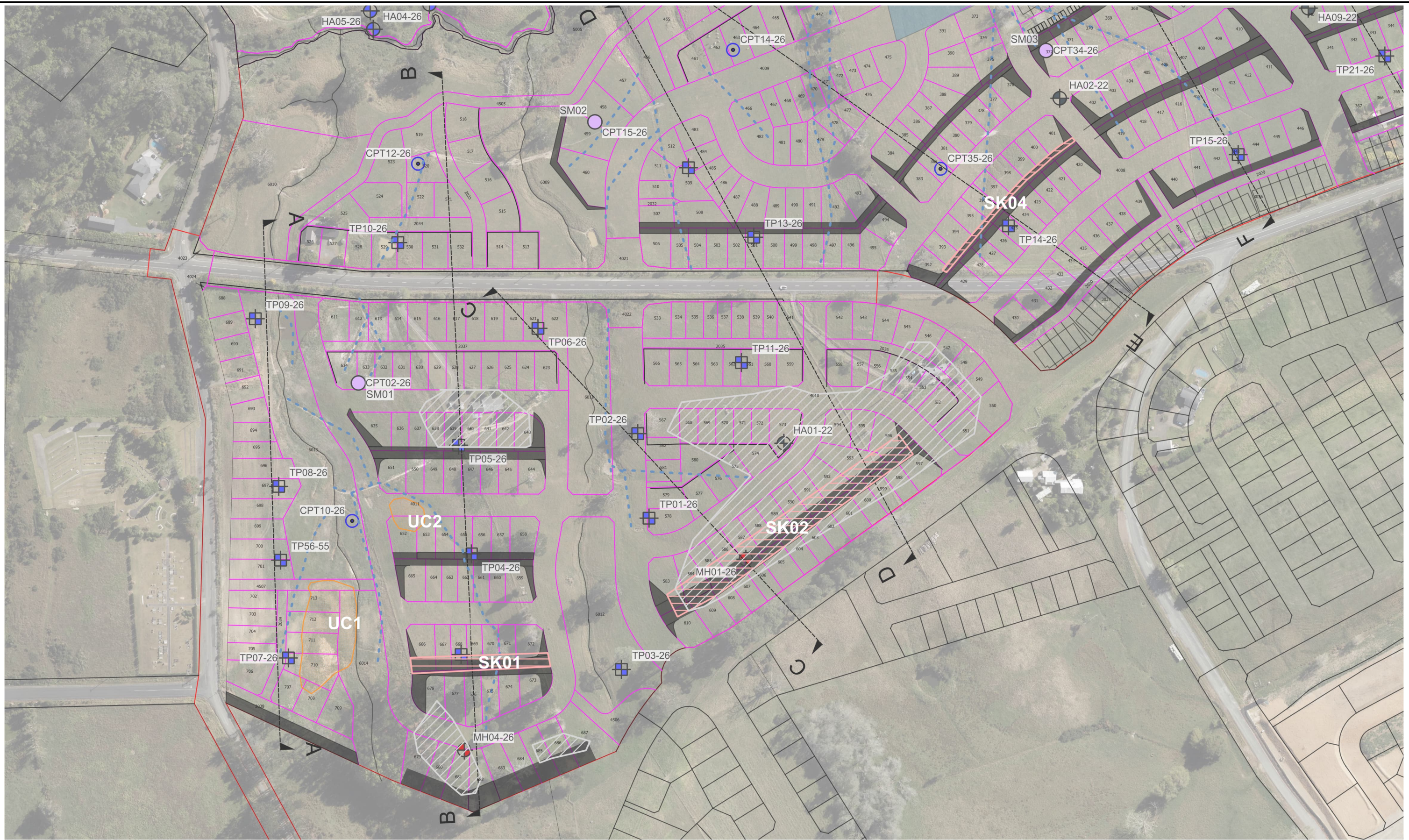


LEGEND:

- Approximate Site Boundary
- Cross Section Locations
- Settlement Monitoring Locations (refer DWG34)
- Shear Key (refer DWG 32)
- Suspected Uncertified Fill
- Rock Mass Exposure
- Trial Embankment (refer DWG 35)
- Underfill Drains (refer DWG 36)
- Preliminary Preload Area

NOTES: 1. BASE MAP ADAPTED FROM AUCKLAND COUNCIL OPEN DATA VIA QGIS
2. DESIGN CONTOURS AND LOT BOUNDARIES FROM WOODS

	CLIENT:	FULTON HOGAN LAND DEVELOPMENT LTD		DRAWN:	CR	PROJECT:	AK26-027
	PROJECT:	MILLDALE STAGES 14-17 FAST TRACK		CHECKED:	SG	DRAWING:	26
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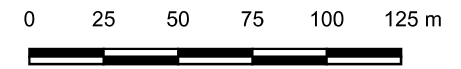


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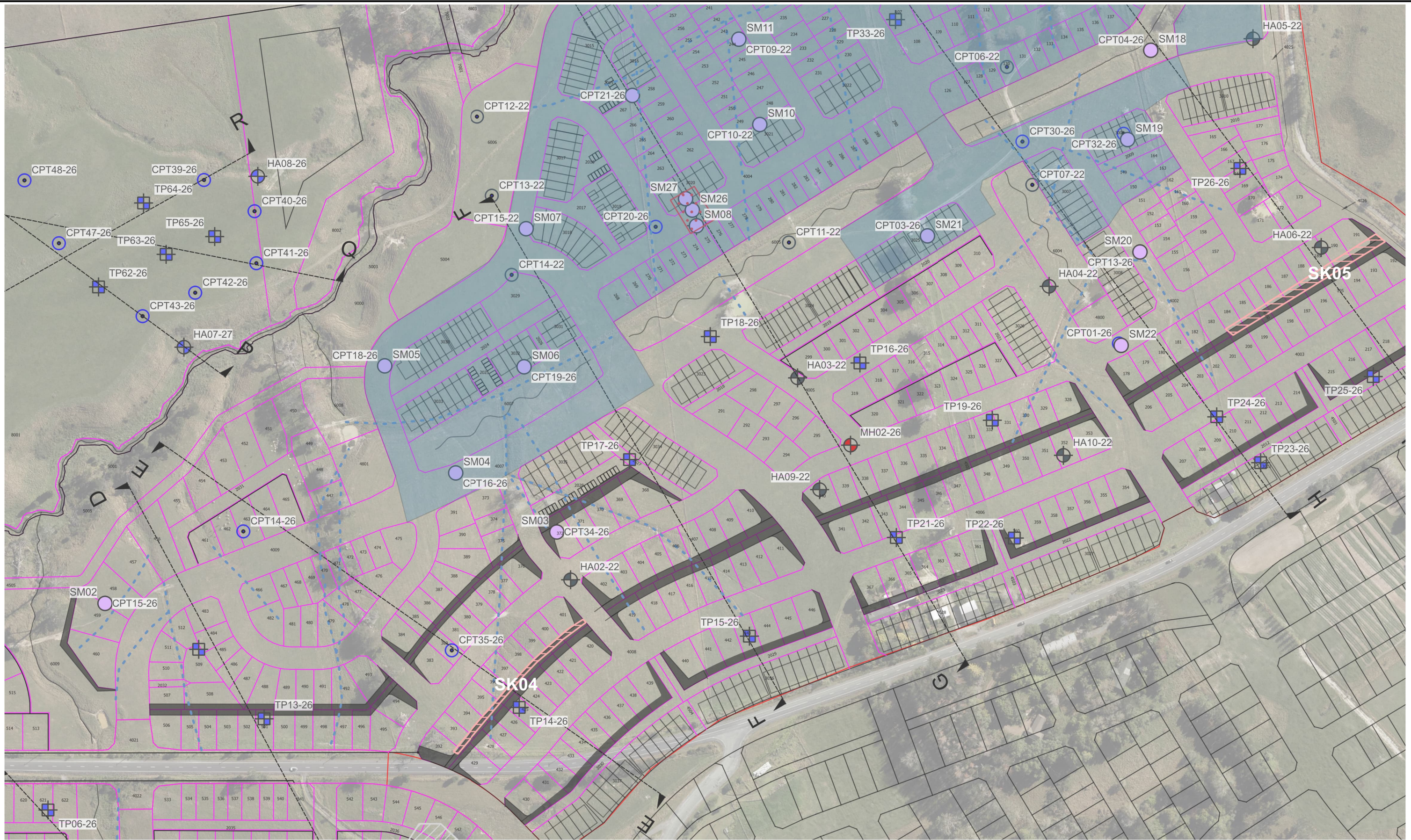
- Approximate Site Boundary
- Cross Section Locations
- Settlement Monitoring Locations (refer DWG34)
- Shear Key (refer DWG 32)
- Suspected Uncertified Fill
- Rock Mass Exposure
- Trial Embankment (refer DWG 35)
- Underfill Drains (refer DWG 36)
- Preliminary Preload Area

NOTES:

1. BASE MAP ADAPTED FROM AUCKLAND COUNCIL OPEN DATA VIA QGIS
2. DESIGN CONTOURS AND LOT BOUNDARIES FROM WOODS



	CLIENT:	FULTON HOGAN LAND DEVELOPMENT LTD		
	PROJECT:	MILLDALE STAGES 14-17 WAINUI EAST		
	TITLE:	PRELIMINARY REMEDIATION 1/5		
	DRAWN:	CR	PROJECT:	AK26-027
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	DATE:	22/07/2026	SHEET:	A3 L



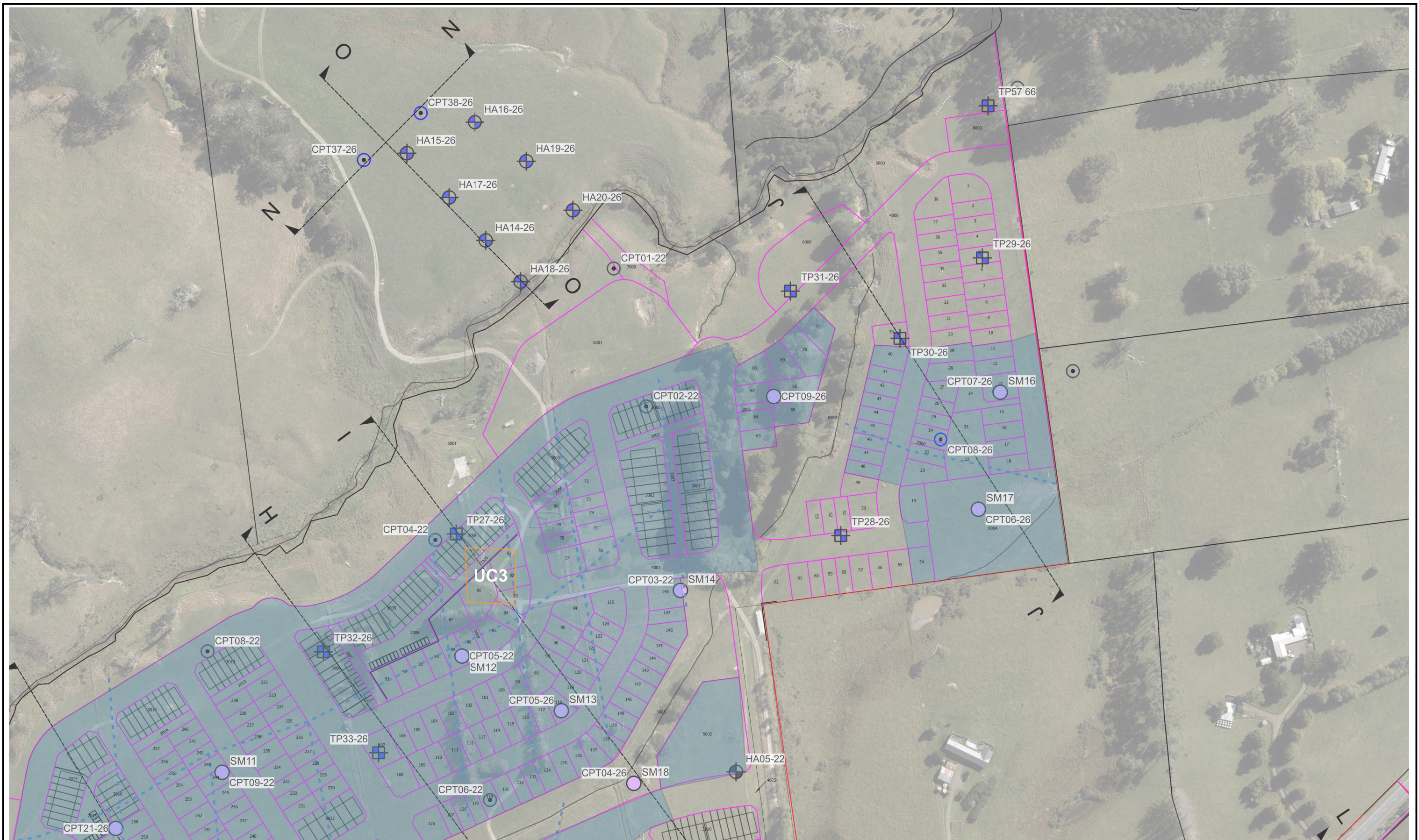
LEGEND:

- Approximate Site Boundary
- Cross Section Locations
- Settlement Monitoring Locations (refer DWG34)
- Shear Key (refer DWG 32)
- Suspected Uncertified Fill
- Rock Mass Exposure
- Trial Embankment (refer DWG 35)
- Underfill Drains (refer DWG 36)
- Preliminary Preload Area

NOTES: 1. BASE MAP ADAPTED FROM AUCKLAND COUNCIL OPEN DATA VIA QGIS
2. DESIGN CONTOURS AND LOT BOUNDARIES FROM WOODS

0 25 50 75 100 125 m

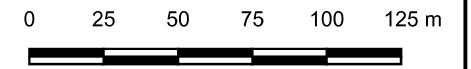
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	PROJECT:	MILLDALE STAGES 14-17 WAINUI EAST		
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	CHECKED:	SG	DRAWING:	29
	REVISION:	0	SCALE:	1:2500
	DATE:	22/07/2026	SHEET:	A3 L



LEGEND:

- Approximate Site Boundary
- Cross Section Locations
- Settlement Monitoring Locations (refer DWG34)
- Shear Key (refer DWG 32)
- Suspected Uncertified Fill
- Rock Mass Exposure
- Trial Embankment (refer DWG 35)
- Underfill Drains (refer DWG 36)
- Preliminary Preload Area

NOTES: 1. BASE MAP ADAPTED FROM AUCKLAND COUNCIL OPEN DATA VIA QGIS
2. DESIGN CONTOURS AND LOT BOUNDARIES FROM WOODS



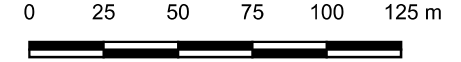
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PROJECT:	MILLDALE STAGES 14-17 WAINUI EAST		
TITLE:	PRELIMINARY REMEDIATION 3/5		
DRAWN:	CR	PROJECT:	AK26-027
CHECKED:	SG	DRAWING:	28
REVISION:	0	SCALE:	1:2500
DATE:	22/07/2026	SHEET:	A3 L



LEGEND:

- Approximate Site Boundary
- Cross Section Locations
- Settlement Monitoring Locations (refer DWG34)
- Shear Key (refer DWG 32)
- Suspected Uncertified Fill
- Rock Mass Exposure
- Trial Embankment (refer DWG 35)
- Underfill Drains (refer DWG 36)
- Preliminary Preload Area

- NOTES:
1. BASE MAP ADAPTED FROM AUCKLAND COUNCIL OPEN DATA VIA QGIS
 2. DESIGN CONTOURS AND LOT BOUNDARIES FROM WOODS



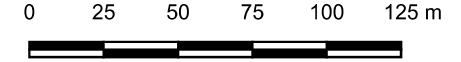
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LEGEND:

- Approximate Site Boundary
- Cross Section Locations
- Settlement Monitoring Locations (refer DWG34)
- Shear Key (refer DWG 32)
- Suspected Uncertified Fill
- Rock Mass Exposure
- Trial Embankment (refer DWG 35)
- Underfill Drains (refer DWG 36)
- Preliminary Preload Area

- NOTES:
1. BASE MAP ADAPTED FROM AUCKLAND COUNCIL OPEN DATA VIA QGIS
 2. DESIGN CONTOURS AND LOT BOUNDARIES FROM WOODS



	CLIENT:	FULTON HOGAN LAND DEVELOPMENT LTD		
	PROJECT:	MILLDALE STAGES 14-17 WAINUI EAST		
	TITLE:	PRELIMINARY REMEDIATION 5/5		
	DRAWN:	CR	PROJECT:	AK26-027
	CHECKED:	SG	DRAWING:	31
	REVISION:	0	SCALE:	1:2500
	DATE:	22/07/2026	SHEET:	A3 L

GENERAL NOTES:

1. GEOGRID TO BE PLACED FLAT OR WITH 1% FALL TO REAR OF WALL, GRID SHOULD BE FREE OF WRINKLES AND LIGHTLY TENSIONED PRIOR TO AND DURING PLACEMENT OF FILL.
2. CARE MUST BE TAKEN TO ENSURE ADEQUATE TENSIONING OF THE GRID WHERE IT IS WRAPPED UP THE FACE.
3. WHERE GRIDS OVERLAP MORE THAN 1 METRE ON CURVED WALLS, 100MM OF FILL SHALL BE PLACED BETWEEN THE GRIDS TO SEPARATE THEM.
4. RUBBER TIRED VEHICLES MAY PASS OVER THE GRID AT SLOW SPEEDS, A MINIMUM OF150mm OF FILL SHALL BE PLACED ON THE GRID PRIOR TO TRAFFICKING BY TRACKED VEHICLES. EXTRA CARE MUST BE TAKEN WHEN USING SHEEPSFOOT TYPE COMPACTORS TO ENSURE THE GRID IS NOT DAMAGED DURING COMPACTION.
5. CONTRACTOR TO ENSURE GRIDS ARE PLACED IN THE CORRECT ORIENTATION. GRID SHOULD BE ROLLED OUT PERPENDICULAR TO THE WALL/ SLOPE FACE.
6. GRID LAYERS MUST BE CONTINUOUS OVER THE DESIGN EMBEDMENT LENGTH. NO JOINS ARE PERMITTED PARALLEL TO THE FACE. LAPS PERPENDICULAR TO THE FACE ARE TO OVERLAP BY 100MM.

INSPECTION HOLD POINTS

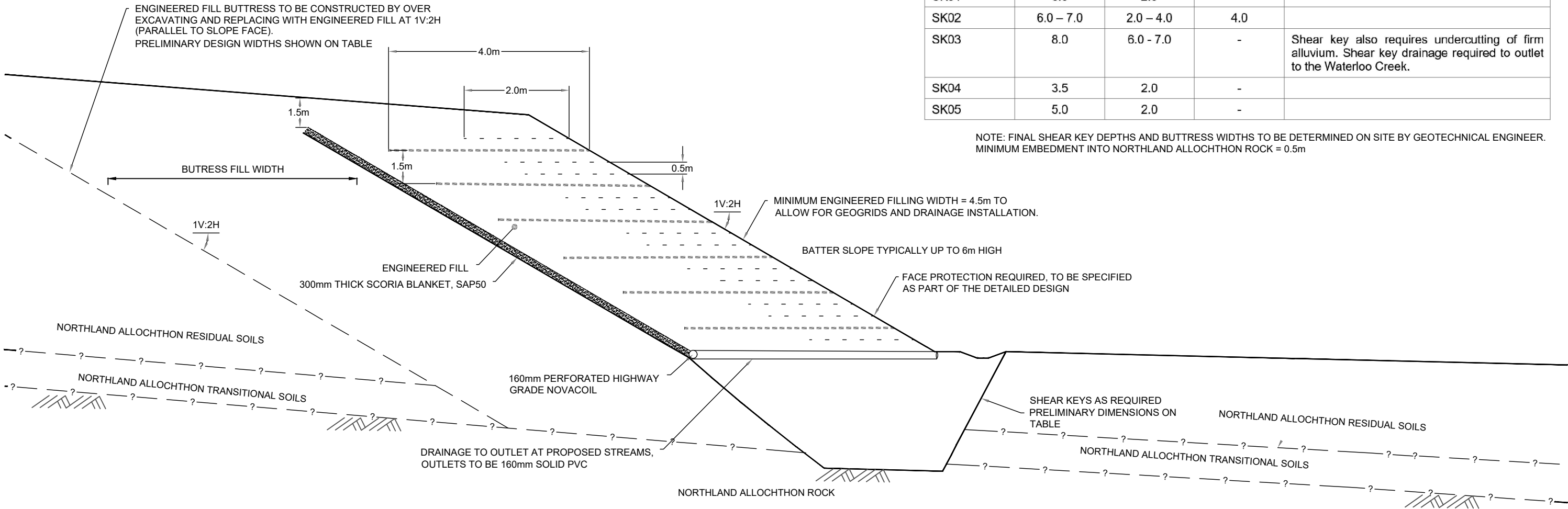
1. BASE OF UNDERCUT
2. DRAINAGE INSTALLATION PRIOR TO BACKFILL
3. GRID LAYER PLACEMENT PRIOR TO BACKFILL
4. COMPACTION TEST ON EACH 1M LIFT PRIOR TO PLACING NEXT GRID LAYER
5. OVERALL LENGTH OF EXCAVATION OPEN AT ANY GIVEN TIME, PLUS HEIGHT OF BACKFILL BEFORE NEXT SECTION OF EXCAVATION COMMENCED TO BE DISCUSSED AND AGREED ON SITE BY CLIENT, CONTRACTOR, ENGINEER TO CONTRACT AND GEOTECHNICAL ENGINEER

KEY	GRID TYPE	EMBEDMENT DEPTH L (m)	VERTICAL SPACING S (m)
=====	PRIMARY	4.0	1.5
- - - - -	SECONDARY	2.0	0.5

PRELIMINARY SHEAR KEY WIDTH AND DEPTHS

Shear reference	key	Base (m)	width	Depth below lowest earthworks surface (m)	RE Slope fill buttress width (m)	Comments
SK01		6.0		2.0	-	
SK02		6.0 – 7.0		2.0 – 4.0	4.0	
SK03		8.0		6.0 - 7.0	-	Shear key also requires undercutting of firm alluvium. Shear key drainage required to outlet to the Waterloo Creek.
SK04		3.5		2.0	-	
SK05		5.0		2.0	-	

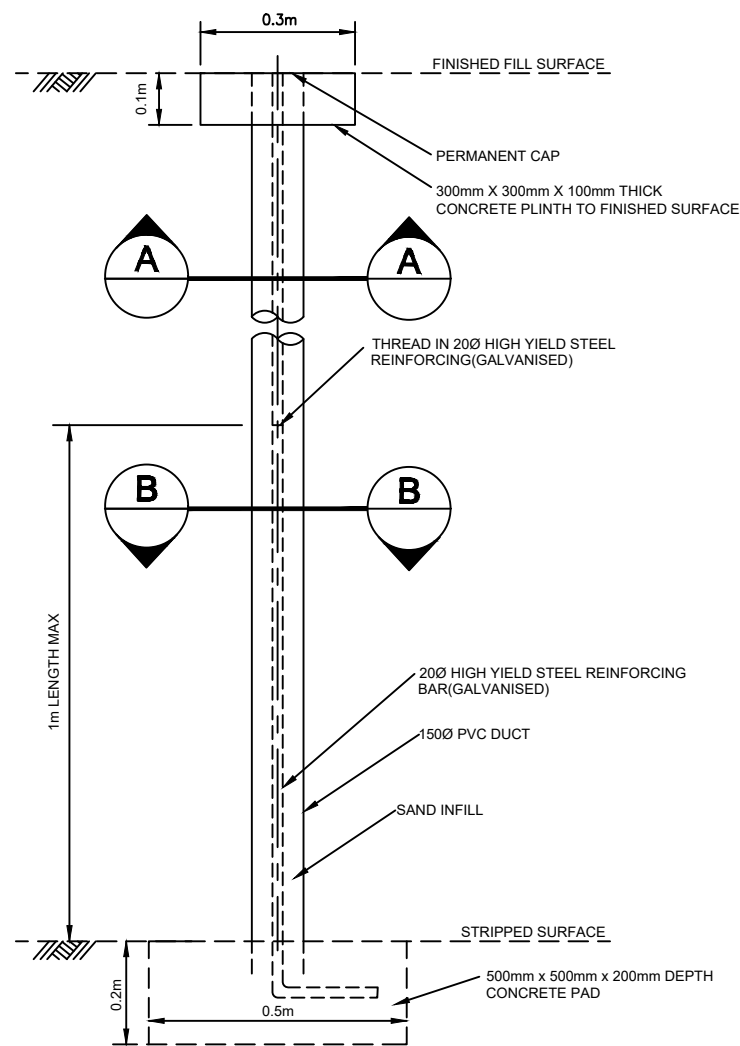
NOTE: FINAL SHEAR KEY DEPTHS AND BUTTRESS WIDTHS TO BE DETERMINED ON SITE BY GEOTECHNICAL ENGINEER. MINIMUM EMBEDMENT INTO NORTHLAND ALLOCHTHON ROCK = 0.5m



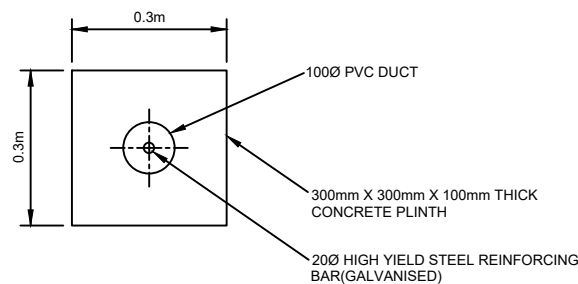
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0	03/06/26	FOR CLIENT REVIEW	CR
REV	DATE	DESCRIPTION	BY



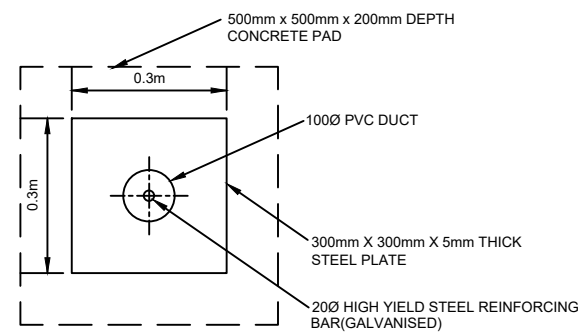
CLIENT:	FULTON HOGAN LAND DEVELOPMENT		DRAWN:	CR	PROJECT:	AK26-027
PROJECT:	MILLDALE NORTH FAST TRACK MILLDALE		CHECKED:	SG	FIGURE:	32
			REVISION:	0	SCALE:	NTS
TITLE:	TYPICAL REINFORCED EARTH BATTER DETAIL		DATE:	03/06/26	SHEET:	A3



SUBGRADE SETTLEMENT MONITORING POINT



CONCRETE PLINTH
SECTION A-A



STEEL PLATE
SECTION B-B

NOTES:

MONITORING FREQUENCY.

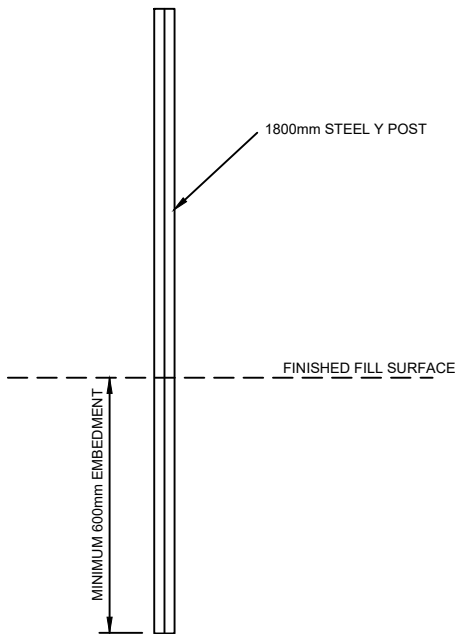
CONSTRUCTION STAGE	SUBGRADE SETTLEMENT MONITORING	SURFACE SETTLEMENT MONITORING
PRIOR TO CONSTRUCTION	AT LEAST 2 BASELINE READINGS (MINIMUM OF 24 HOURS SHALL APPLY BETWEEN READING SETS)	NOT REQUIRED
DURING FILLING	ONCE PER WEEK OR PER 0.5M DEPTH (WHICHEVER IS MORE FREQUENT) INCLUDING MEASUREMENT OF ADJACENT FILL LEVEL	
COMPLETION OF FILLING	FORTNIGHTLY FOR TWO MONTHS MONTHLY FOR SIX MONTHS	MONTHLY FOR SIX MONTHS

THE FREQUENCIES ABOVE MAY BE DECREASED OR INCREASED BY THE GEOTECHNICAL ENGINEER DEPENDING ON THE RESULTS OF THE MONITORING.

SETTLEMENT AND RETAINING WALL MARKERS SHALL BE LEVELLED BY PRECISE SURVEY TO AN ACCURACY OF $\pm 2\text{MM}$ AT THE FREQUENCIES OUTLINED ABOVE. ALL SURVEY DATA MUST BE PROVIDED TO THE GEOTECHNICAL ENGINEER AS SOON AS POSSIBLE FOLLOWING COMPLETION.

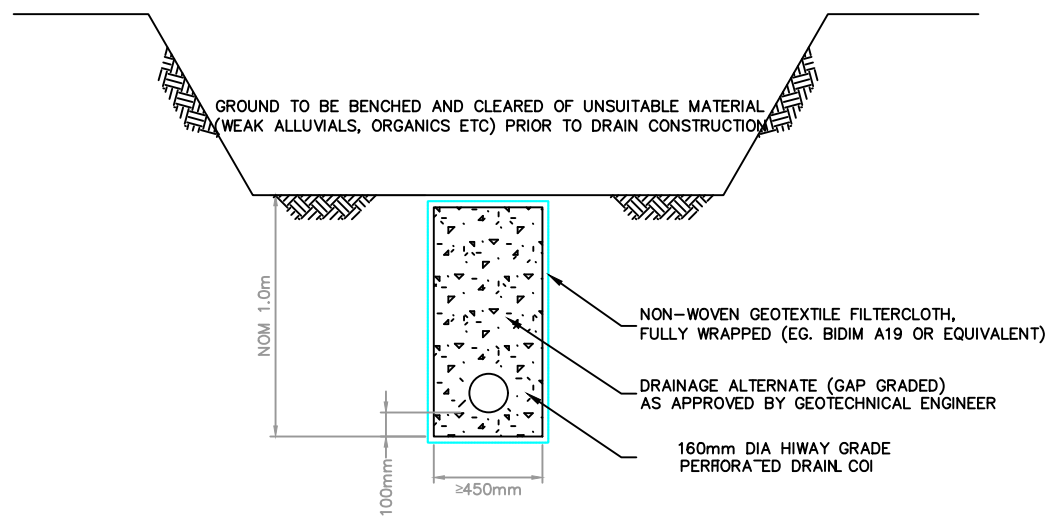
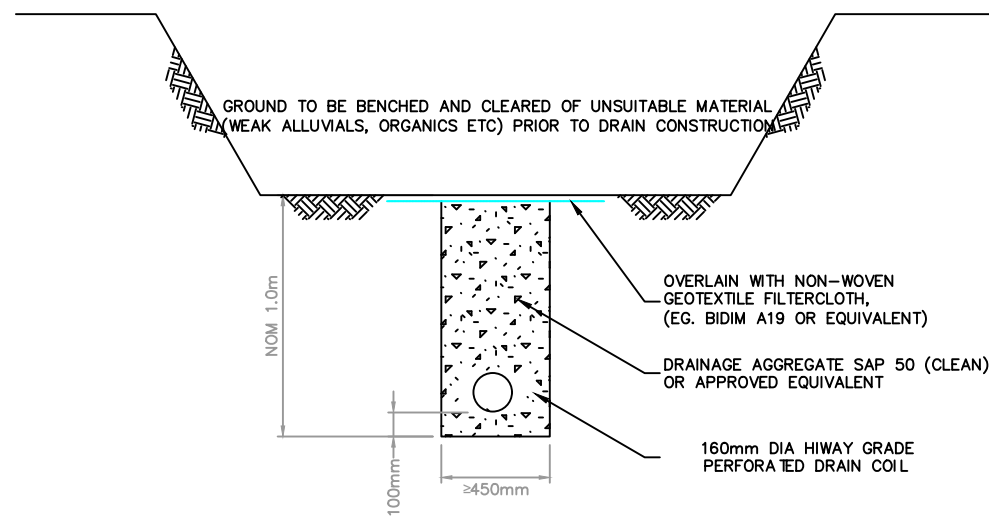
THE SETTLEMENT MARKERS (TOP OF UPSTAND) MUST BE PRECISELY LEVELLED AND THE PRECISE LENGTH OF THE UPSTAND MEASURED AT THE TIME OF INSTALLATION PRIOR TO ANY FILL PLACEMENT

IT IS THE CONTRACTORS RESPONSIBILITY TO ENSURE THE INTEGRITY OF THE MONITORING PLATES IS MAINTAINED DURING THE WORKS AND UNTIL THE GEOTECHNICAL ENGINEER HAS INSTRUCTED THE PLATES CAN BE REMOVED.



SURFACE SETTLEMENT MONITORING POINT

	CLIENT: FULTON HOGAN LAND DEVELOPMENT LTD	DRAWN: SG	PROJECT: AK26-027
	PROJECT: MILLDALE STAGES 14-17	CHECKED: CR	FIGURE: 33
	TITLE: SETTLEMENT MONITORING POINT DETAILS	REVISION: 0	SCALE: NTS
		DATE: 04/06/26	SHEET: A3



CLIENT:	FULTON HOGAN LAND DEVELOPMENT LTD		DRAWN:	SG	PROJECT:	AK26-027
PROJECT:	MILLDALE STAGES 14-17		CHECKED:	CR	FIGURE:	34
TITLE:	SUBSOIL DRAINAGE DETAILS		REVISION:	0	SCALE:	NTS
			DATE:	04/06/26	SHEET:	A3