



GEOTECHNICAL INVESTIGATION REPORT

Milldale Stages 14-17 Water Infrastructure Facility

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

31 July 2026

STATEMENTS OF QUALIFICATIONS AND EXPERIENCE

Chris Ritchie

My full name is Christopher John Hugh 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

My full name is Samuel Edward 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.

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).

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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 Water Infrastructure Facility 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
12 June 2026	A	Internal Draft
18 June 2026	0	Draft for Review
31 July 2026	1	Final

	Name	Position	Signature
Prepared by	Chris Ritchie	Principal Geotechnical Engineer, CPEng	
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 a proposed water infrastructure facility near Milldale, Auckland.

The water infrastructure plant forms part of the wider Milldale Stage 14-17 project, which 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.

This report has been prepared in support of the substantive application for the Milldale Stages 14-17 (Milldale North Project under the Fast-track Approvals Act 2024 (FTAA).

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 and ecological restoration, and associated enabling works.

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 Water Infrastructure Facility as part of the wider project proposal. 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. Further geotechnical investigation, analysis and design will be required prior to detailed design of the water infrastructure plant, to support a building consent application.

2 SITE DETAILS

The subject site named Te Korowai o Maraeariki is located to the north of the Ōrewa river and is zoned Rural Production under the AUP. This area is characterised by more undulating to steep terrain, with a series of ridgelines separated by incised gullies extending from the Ōrewa River. The land is predominantly managed pasture interspersed with streams and natural wetlands, together with pine plantations, exotic vegetation and gorse scrub. Relatively large areas of indigenous vegetation are also present, including a Significant Ecological Area – Terrestrial Overlay in the north-western portion of the site.

3 PROPOSED DEVELOPMENT

To the north of the Ōrewa river within Te Korowai o Maraeariki, the project includes private water and wastewater infrastructure if required, ecological restoration, as well as walking and cycling trails to enhance recreation and amenity. The aspects of the proposal relevant to this report include:

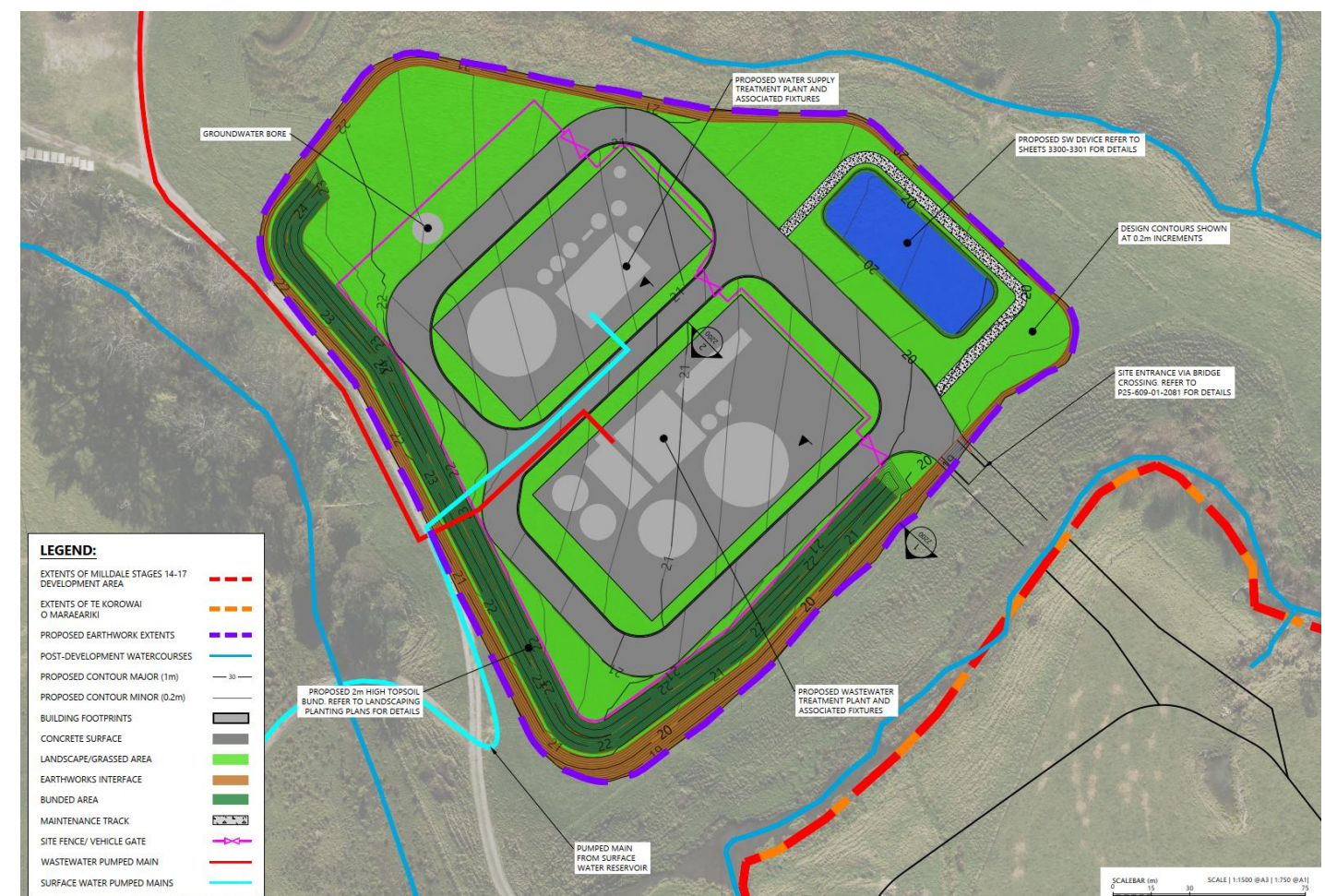
- The water infrastructure facility includes a wastewater treatment plant, designed by Apex Water to accommodate 1425 new dwellings
- The wastewater treatment plant comprises several buildings and large tanks up to 7m in height
- The facility also includes water supply in the form of a groundwater bore and associated treatment buildings

Earthworks design by Woods depicts fills up to 3m in height to form a batter on the south western and southern boundary.

Figure 1: Site Location Plan



Figure 2: Proposed Plant Layout



4 DESKTOP STUDY

4.1 Related Documents

The following documents were reviewed during preparation of this report:

- Geotechnical Investigation for Milldale Stages 14-17, prepared by Gibb Ritchie Geotechnical Engineers Ltd referenced AK26-027AC Rev 0, dated 5 June 2026.
- Geotechnical Investigation for Milldale North, prepared by CMW Geosciences referenced AKL2024-0257AB Rev 3, dated 25 March 2025.

No specific investigations for this site are contained within these reports, however testing data on the geotechnical units has been incorporated into the ground model for this site.

4.2 Existing Information

No existing geotechnical information is present at the site.

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. In summary the area has been used for farming from the 1940s with minimal changes occurring.

4.4 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 East Coast Bays Formation of the Waitemata Group. A description of these geological units can be found in the table below.

Table 1: 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, dependent 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.
East Coast Bays Formation of the Waitemata Group	
Formation	Sea floor deposits that have been compressed, then uplifted, faulted, locally folded or tilted and then weathered
Typical Composition, Weathering/ Layering and Variability	Where undisturbed, weathers to a consistent profile with 3 generalised layers: Residual soils at the surface, overlying a less weathered (typically dark grey and very stiff/ hard) transition with relict bedding, that itself overlies A layered mudstone/ sandstone sequence of rock material. Can demonstrate a stepped rock surface profile due to the faulting, with variations in weathering depths across faults and low-strength bedding planes near the rock surface. Rock material strength is typically low but some beds (eg Parnell Grit) are significantly stronger

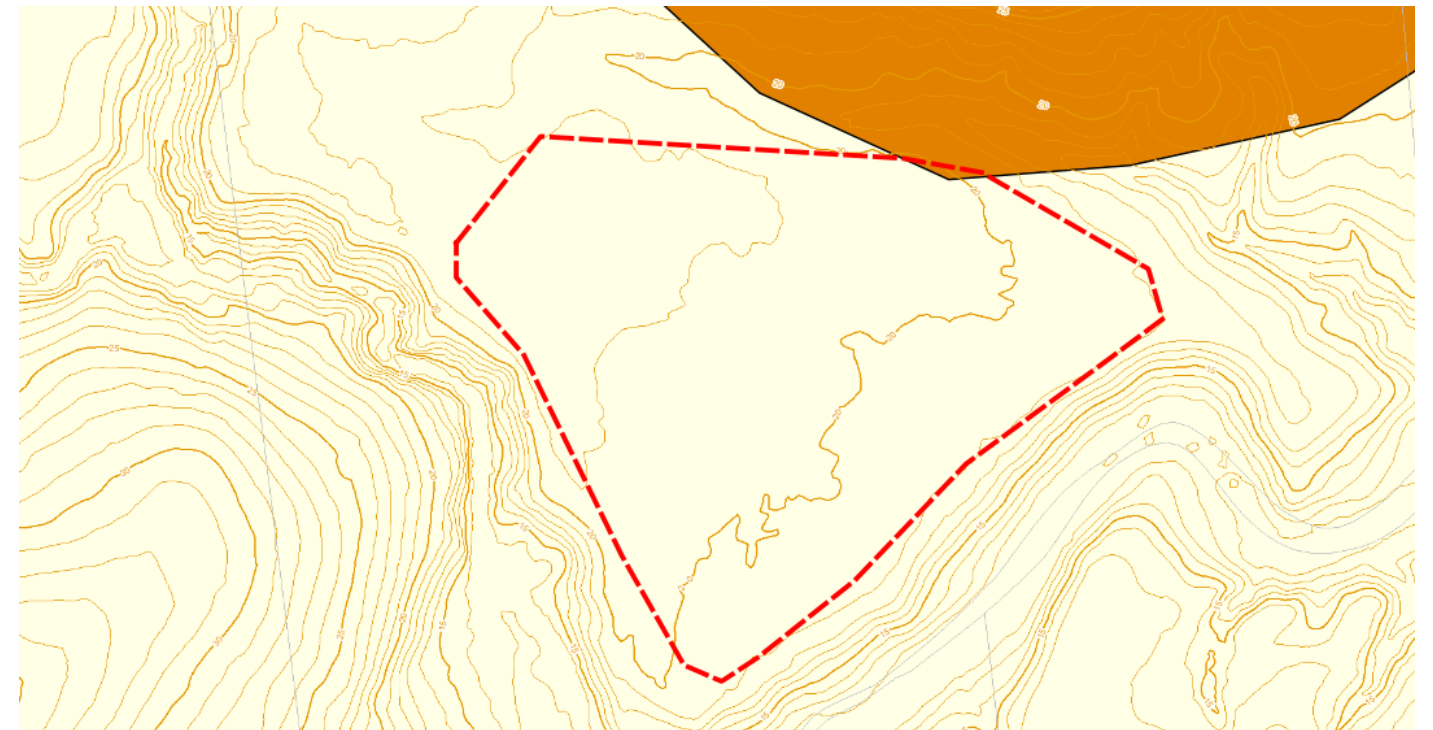
Behaviour, Instability Mechanism(s) and Principal Potential Geohazards	High shrink/ swell/ expansivity of near-surface clays Shallow circular instability and soil creep in moderately graded slopes Deep seated block failures governed by unfavourable rock defects and bedding orientations and inclinations and trapped groundwater deposits.
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4.5 Geomorphology

The site geomorphology can be broadly described by the following:

- The site occupies a broad alluvial terrace which is bound by watercourses to the west and east and the Orewa River to the south.
- This terrace is near flat, very gently sloping from RL21 in the northeast to RL18 in the southeast.
- The watercourse to the west is within a gully that falls at approximately 1V in 3H to 1V in 4H.
- The Orewa Riverbank falls to the southeast at approximately 1V:4H.
- To the north of the site, the topography rises to steep slopes, likely underlain by Waitemata Group materials.

Figure 3 Geology Map from QMAP



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

5 INVESTIGATIONS AND RESULTS

5.1 Field Investigation

The scope of fieldwork completed is shown below:

Table 2 Site Investigation

Test IDs	Test Type	No. of Tests	Depth Range (m)
HA14 to 20-26	Hand Auger Borehole	7	4
CPT37,38-26	Cone Penetration Tests	2	13-16.5

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 as Drawing 35.

5.2 Groundwater

During the investigation, completed in early winter conditions (June 2026) groundwater was encountered at depths ranging from 1.7m to > 4m below existing ground levels.

The boreholes were dipped the following week with levels ranging from 0.39m to >4m these may have been affected by surface water inflow.

6 GROUND MODEL

6.1 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.

Two geological cross sections (Section O and N) have been produced at the locations noted on Figure 4 and shown on Drawings 36 and 37.

Table 3: Ground Model

Geology (Map Code)	Geotechnical Unit	Material Description	Geomorphology and History	Key Geotechnical Hazard
Middle to Late Pleistocene Hill and River Deposits (IQa)	Stiff to Very Stiff Alluvium	Stiff to very stiff low plasticity clays and silts.	Elevated areas of alluvial soils likely deposited when sea levels were much higher. Erosive surface incised by the Orewa River and Waterloo Stream.	Soils expected to be over consolidated due to previous loading. Soils compressible under high fill / building loading. Liquefaction unlikely due to age of deposits.
East Coast Bays Formation Waitemata Group (ECBF)	ECBF Rock Mass	Extremely to very weak siltstone and sandstone.	Not exposed at surface within the project area, however inferred from CPT trace and close boundary to mapped ECBF.	Potential for highly expansive soils. Bedding shears can be present near rock transition may cause instability when exposed.

6.2 Recommended Geotechnical Parameters

Table 4 Preliminary Geotechnical Design Parameters

Geotechnical Unit	Testing Results			Unit Weight	Drained	Undrained
	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)
Stiff to Very Stiff Alluvium	50 – 200 (75)	1 – 2	17	3	28	65
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, CPT correlations)

S_u = undrained shear strength (lower bound from vane shear testing and CPT correlations)

Figure 4 Site Investigation Plan and Cross Section Locations



7 GEOHAZARDS ASSESSMENT

A full risk assessment in accordance with Auckland Council Code of Practice for Land Development Chapter 2 Geotechnical Engineer can be found in Appendix G. From this assessment the key geohazards for this project are assessed as:

- Static settlement of compressible alluvial soils due to large building loads, including tanks.
- Bearing capacity of alluvial soils for large tank structures.
- Expansive soils damaging future foundations.

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

7.1.1 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 5 Risk Assessment

Hazard	Likelihood	Consequence	Unmitigated Risk	Comment
Flooding	Assessed by others			
Landslip	Rare	Moderate	Low	Refer to Section 7.5 for assessment
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 7.3 for detailed assessment.
Tsunami	Assessed by others			

7.2 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 6 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

7.3 Liquefaction

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

Figure 5: 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).
- Thin-layer correction was applied based on the rate of change of the soil behaviour type index value (I_c) method

Table 7 Calculated Liquefaction Settlements

CPT No.	SLS Settlement (mm)	Total ² ULS Settlement (mm)	Depth to Liquefiable Layer (m)
CPT37-26	0	25	5, 6, 12
CPT38-26	0	<10	5
Note: All settlement and depths based on existing ground profile ² Total ULS settlements are based on the full depth of the CPT trace with a depth weighting factor applied.			

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). Results can be found in Appendix C.

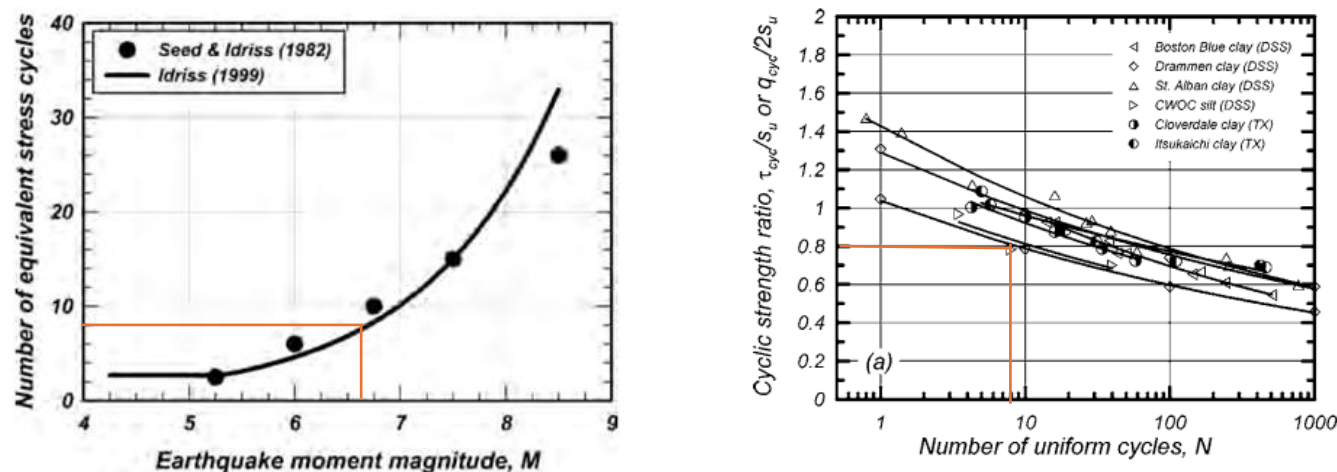
7.4 Cyclic Softening

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 Boulanger⁵ and Idriss⁶. This correlates earthquake magnitude to the estimated number of equivalent stress cycles and then correlates number of cycles to a cyclic shear strength ratio (Figure 6).

Based on the above assessment, 2 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 should be considered during foundation design.

Figure 6: Relationship Between Earthquake Magnitude and Mean Number of Uniform Stress Cycles and Relationship Between cyclic Strength Ratio and Number of Uniform Stress Cycles



7.5 Slope Stability

Slope stability analysis has been carried out for two cross sections which were selected as areas containing the largest tanks and the Orewa riverbank.

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 (ACCOPs).

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	

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

Table 9 Existing Section Slope Stability Factors of Safety

Section	Existing Slope Stability Minimum Factor of Safety		
	Normal GW	Extreme GW	ULS Seismic
N-N	1.4	1.3	1.5
O-O	1.3	1.3	1.3

Table 10 Proposed Sections Factor of Safety Requirements

Section	Proposed Slope Stability Minimum Factor of Safety		
	Normal GW	Extreme GW	ULS Seismic
N-N	1.4*	1.3*	1.3
O-O	1.3*	1.3	1.1
* Factor of safety of >1.5 or >1.3 achieved within platform			

Based on the stability modelling, the water treatment facility is sufficiently offset from the steeply sloping ground and meets the factor of safety requirements set out by ACCOPs. Outputs from the stability modelling can be found in Appendix D.

7.5.1 PC120 Landslide Assessment

A landslide risk assessment in accordance of Appendix 24 of Plan Change 120 was carried out and be found in Appendix E. In summary:

- Large scale landslide susceptibility is mapped as very low within the footprint of the facility,
- Shallow landslide susceptibility is mapped as generally very low.
- To determine run-out susceptibility, the highest landslide susceptibility class mapped within 150m was reviewed. This shows very high shallow landslide susceptibility on the neighbouring slopes above the facility and along the nearby water courses.

⁵ 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.

⁶ Idriss, I. M. and Boulanger, R. W. (2008) Soil Liquefaction During Earthquakes. Monograph 12, Earthquake Engineering Research Institute.

We undertook a Method 1 and Method 2 risk assessment for three hazard scenarios. The result was a low landslide risk for the three scenarios including a combined risk using method 2. Based on the desktop assessment, we consider that landslide hazard result is Low (acceptable) under Appendix 24.

7.6 Static Settlement

Proposed building and tank loads will induce settlements within the underlying subsoils. An initial screening exercise using CPeT-IT was undertaken to determine the risk of static settlement for these structures.

7.6.1 CPT Method

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 settlements, three types of plant loads were assessed for each CPT, to determine total and potential differential settlement. These were:

- 4.2m high tanks such as aeration, post anoxic and screen influent storage tanks (assumed uniformly distributed load of 45kPa)
- 3m high proposed tanks such as clearwater and WAS tanks (assumed uniformly distributed load of 30kPa)
- Other plant structures assumed to have a uniformly distributed load of 20kPa
- Total settlements comprised an assumed 1 year for primary settlement and 49 years for secondary creep.
- We have assumed that fill settlements would have reached T90 prior to construction of the tanks.

Table 11 CPT Settlement Estimations

Structure Assessed	Total Settlement (mm) CPT37-26	Total Settlement (mm) CPT38-26	Assumed Structure Width (m)	Potential Differential Settlement (slope)
4.2m high tank (45kPa UDL)	60	80	9.2	1 in 450
3m high proposed WAS / clearwater / permeate tank (UDL 30kPa)	60	100	35	1 in 875
Other plant structures (UDL 20kPa)	40	60	10	1 in 500

It is likely that total settlements experienced by these structures is beyond acceptable limits, in particular where connections are made between pipework which is unlikely to settle with the tanks / buildings. Recommendations for the mitigation of this hazard is made in Section 8.2.

Outputs from CPeT-IT can be found in Appendix F.

7.6.2 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 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 (Cv) 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 12 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 Cv 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.

8 RECOMMENDATIONS

8.1 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 water infrastructure plant, however heavily loaded structures including tanks are likely to undergo consolidation settlement over their design life and the stiff alluvial soils may not provide adequate bearing capacity without ground improvement.

This, along with earthworks and civils works recommendations are made in the following sections.

8.2 Settlement Mitigation

8.2.1 Foundation and Ground Improvement Options

To mitigate post construction static ground settlements to acceptable 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 prior to foundation construction;
- Undertake ground improvement beneath the embankment footprint, such as stone columns, soil mixed columns, CFA piles or rigid inclusions to transfer loads from the heavily loaded buildings to more competent underlying soil/rock at depth.
- Incorporate structural piles into the foundation design of heavily loaded structures to transfer vertical load to the underlying rock mass.
- Flexible connections between pipework and plant buildings to accommodate differential settlement.

All these options require further geotechnical investigation and design prior to building consent.

8.3 Earthworks

All earthwork activities must be carried out in general accordance with the requirements of NZS 4431:2022⁷ and the requirements of the Auckland Council Infrastructure Development Code under the guidance of a Chartered Geotechnical Engineer. A geotechnical works specification has been provided with the main subdivision GIR (referenced AK26-027AB) which provides 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 13: Key Construction Risks

Item	Mitigation Measures
Excavatability	Excavations likely to comprise alluvial soils 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 bearing capacity failures on alluvial soils.

8.4 Civil Works

Table 14: Key Civil Inputs

Item	Value	Comment
Subgrade CBR	3-5% Stiff Alluvium 5-6% Engineered Fill	
Typical Topsoil Depths	100 – 300mm	

8.5 Foundations

The following table provides typical foundation design parameters, however we anticipate that all buildings here will require specific geotechnical foundation design and further investigations.

Table 15: Anticipated Shallow Foundation Details

Item	Methodology	Value	Comment
Geotechnical Ultimate Bearing Capacity (GUBC) for shallow foundations		Case by case basis	We anticipated that bearing pressures will vary significantly between the tanks and structures. Further investigation is required prior to providing bearing pressures and associated settlement characteristics. Must include provisions for cycle softening.
Expansive Soil Site Class	AS2870	M (moderate) to H (high)	
Strength reduction factors	B1/VM2 ⁸	0.8 0.5	Load combinations involving earthquake overstrength All other load combinations
Seismic Site Class(es)	NZS1170	C	CPT refusal on inferred ECBF rock.

Table 16: Preliminary Pile Design

The following values are preliminary only and have been determined using correlations out of CPT-IT.

Item	Value	Comments
Geotechnical Ultimate End Bearing Capacity (kPa)	450kPa 1500kPa	Stiff Alluvium ECBF rock mass
Geotechnical Ultimate Side Adhesion (kPa)	25kPa 80kPa	Stiff Alluvium ECBF rock mass
Soil Parameters	Refer to Table 3	

8.6 Certification

Given the extent of earthworks and purpose of the development, we do not recommend a Geotechnical Completion Report (GCR) for this area. Certification of this facility including buildings should be covered by a series of Producer Statement 4s for the plant buildings and tanks.

⁷ Standards New Zealand (2022) Engineered fill construction for lightweight structures, NZS 4431:2022

⁸ Ministry of Business, Innovation and Employment (2025) Acceptable Solutions and Verification Methods for NZ Building Code Clause B1 Structure, B1/VM2, Second Edition

9 FURTHER WORK

The following further work is recommended as the project progresses:

- Further geotechnical investigation for the detailed design of plant buildings and tanks.
- Detailed geotechnical design for foundations, particularly of large tank structures where differential settlement pose a significant risk.
- During construction, regular inspections and testing will be required.

10 CLOSURE

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 Water Infrastructure Facility 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 Water Infrastructure Facility 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.

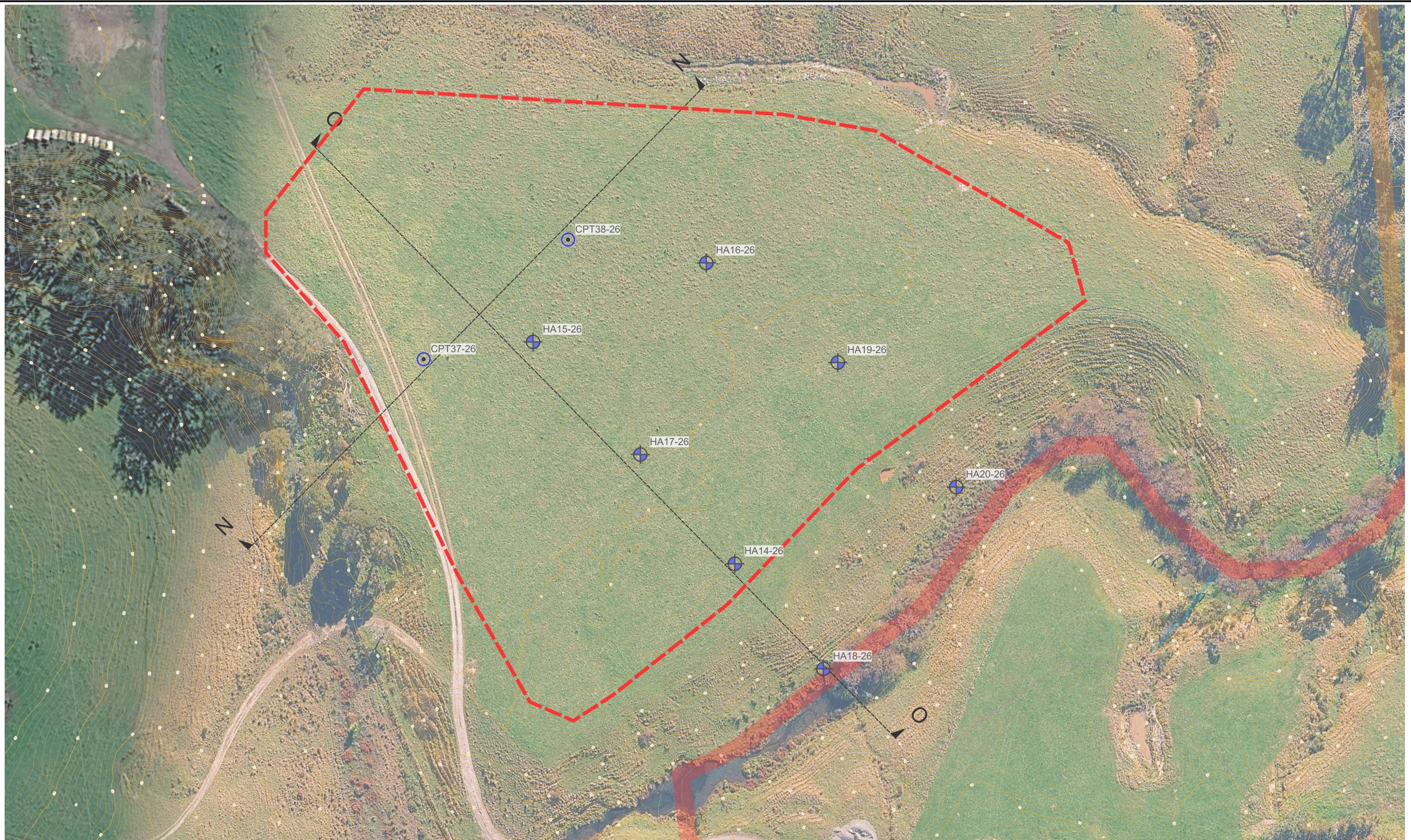
Environmental matters

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:

- SITE BOUNDARY
- ▲ CROSS SECTION LOCATION

Gibb Ritchie Geotechnical Engineers Investigations

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

Existing Investigations

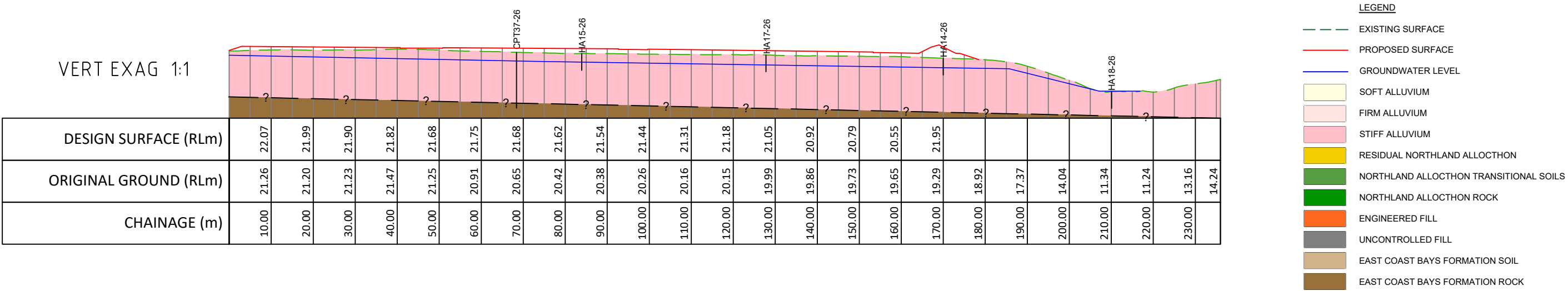
- ⊕ CMW HAND AUGER (HA) LOCATION
- ⊕ CMW TEST PIT (TP) LOCATION
- ⊕ CMW MACHINE HOLE (MH) LOCATION
- ⊙ CMW CPT LOCATION

NOTES: 1. BASE MAP ADAPTED FROM AUCKLAND COUNCIL OPEN DATA VIA QGIS

0 25 50 m



CLIENT:	FULTON HOGAN LAND DEVELOPMENT LTD		
PROJECT:	MILLDALE STAGES 14-17 WAINUI EAST		
TITLE:	SITE INVESTIGATION PLAN		
DRAWN:	CR	PROJECT:	AK26-027
CHECKED:	SG	DRAWING:	35
REVISION:	A	SCALE:	1:1000
DATE:	17/06/2026	SHEET:	A3 L

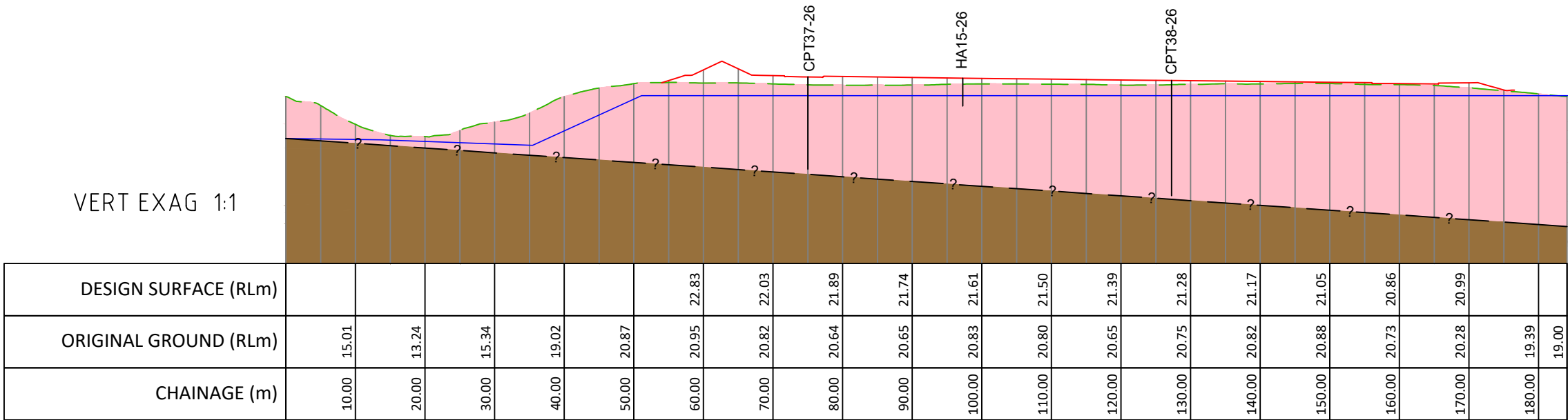


GEOLOGICAL LONG SECTION O



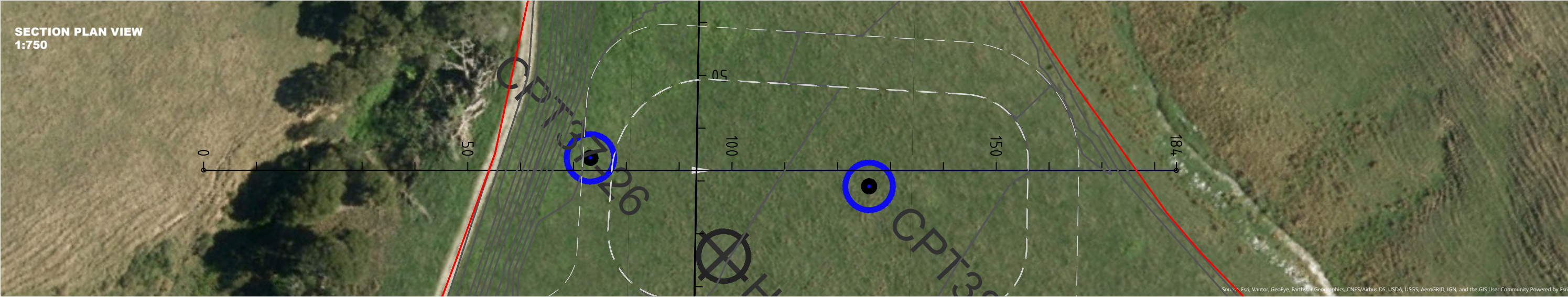
A	15/04/26	FOR INTERNAL REVIEW	CR
0	31/07/26	FOR CLIENT ISSUE	CR
REV	DATE	DESCRIPTION	BY

CLIENT:	FULTON HOGAN LAND DEVELOPMENT		DRAWN:	IS	PROJECT:	AK26-027
PROJECT:	MILLDALE NORTH WASTEWATER TREATMENT PLANT		CHECKED:	SG	FIGURE:	36
			REVISION:	0	SCALE:	1:1000
TITLE:	GEOLOGICAL LONG SECTION O		DATE:	08/07/6	SHEET:	A3



- 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

GEOLOGICAL LONG SECTION N



A	8/07/26	FOR INTERNAL REVIEW	CR
0	31/07/26	FOR CLIENT ISSUE	CR
REV	DATE	DESCRIPTION	BY

	CLIENT:	FULTON HOGAN LAND DEVELOPMENT		DRAWN:	IS	PROJECT:	AK26-027
	PROJECT:	MILLDALE NORTH WASTEWATER TREATMENT PLANT		CHECKED:	SG	FIGURE:	37
				REVISION:	0	SCALE:	1:750
	TITLE:	GEOLOGICAL LONG SECTION N		DATE:	08/07/6	SHEET:	A3

APPENDIX B

Investigation Records

Coordinate System : EPSG:2193

Easting (m) : 1746948.05

Northing (m) : 5948541.17

Ground Elevation : 17.00 (m)

Total Depth : 4 m BGL

Drill Rig : Hand Auger

Driller Supplier : Gibb Ritchie Geotechnical Engineers Ltd

Logged By : Ilhan Serif

Reviewed By :

Date : 27/05/2026

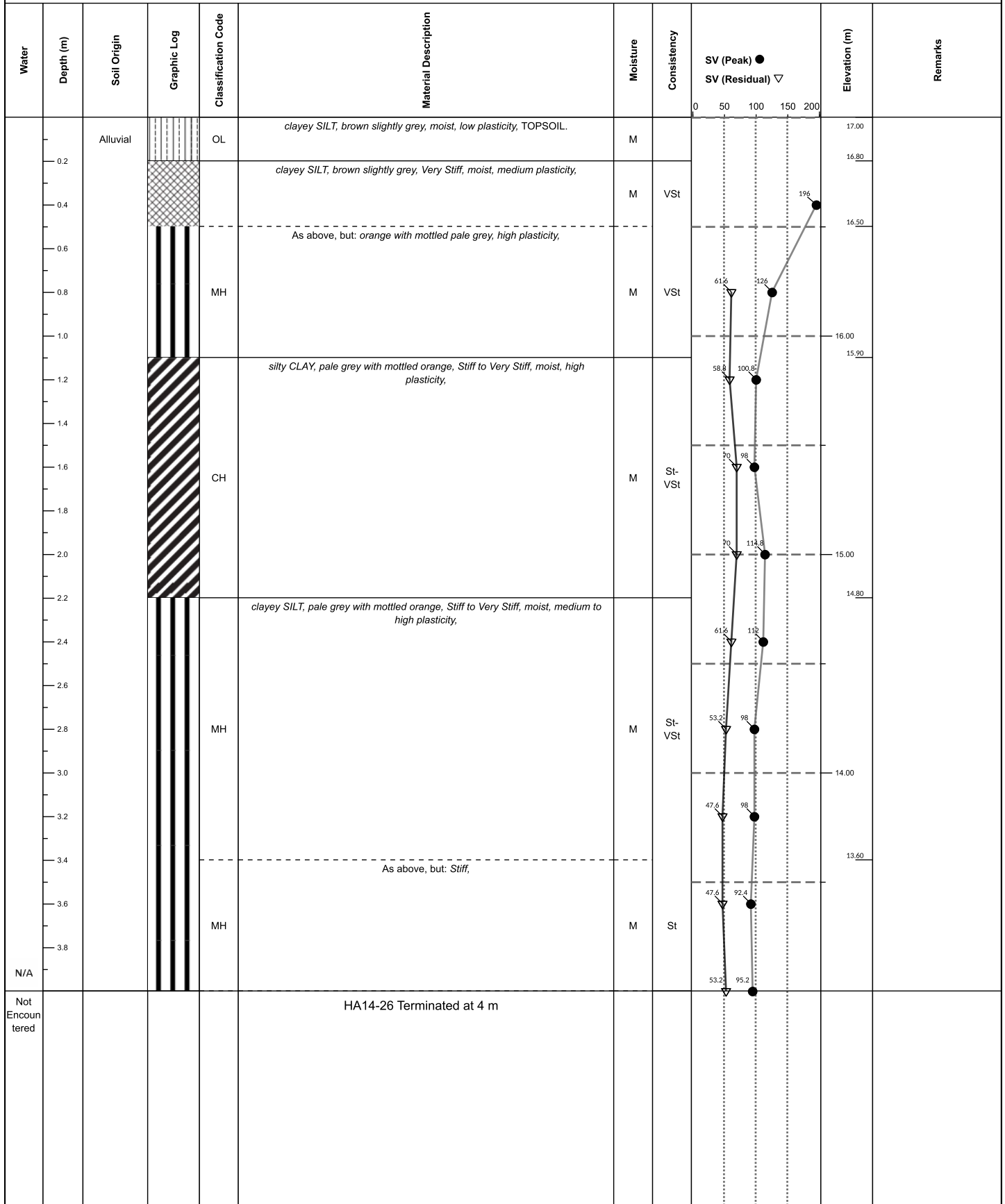
Job Number : AK26-027

Client : Fulton Hogan Land Development Ltd

Project : Milldale North

Location : 522 Wainui Road, Wainui 0992, New Zealand

Loc Comment :



Loc Comment :



Coordinate System : EPSG:2193

Easting (m) : 1746936.90

Northing (m) : 5948621.59

Ground Elevation : 18.00 (m)

Total Depth : 4 m BGL

Drill Rig : Hand Auger

Driller Supplier : Gibb Ritchie Geotechnical Engineers Ltd

Logged By : Ilhan Serif

Reviewed By :

Date : 25/05/2026

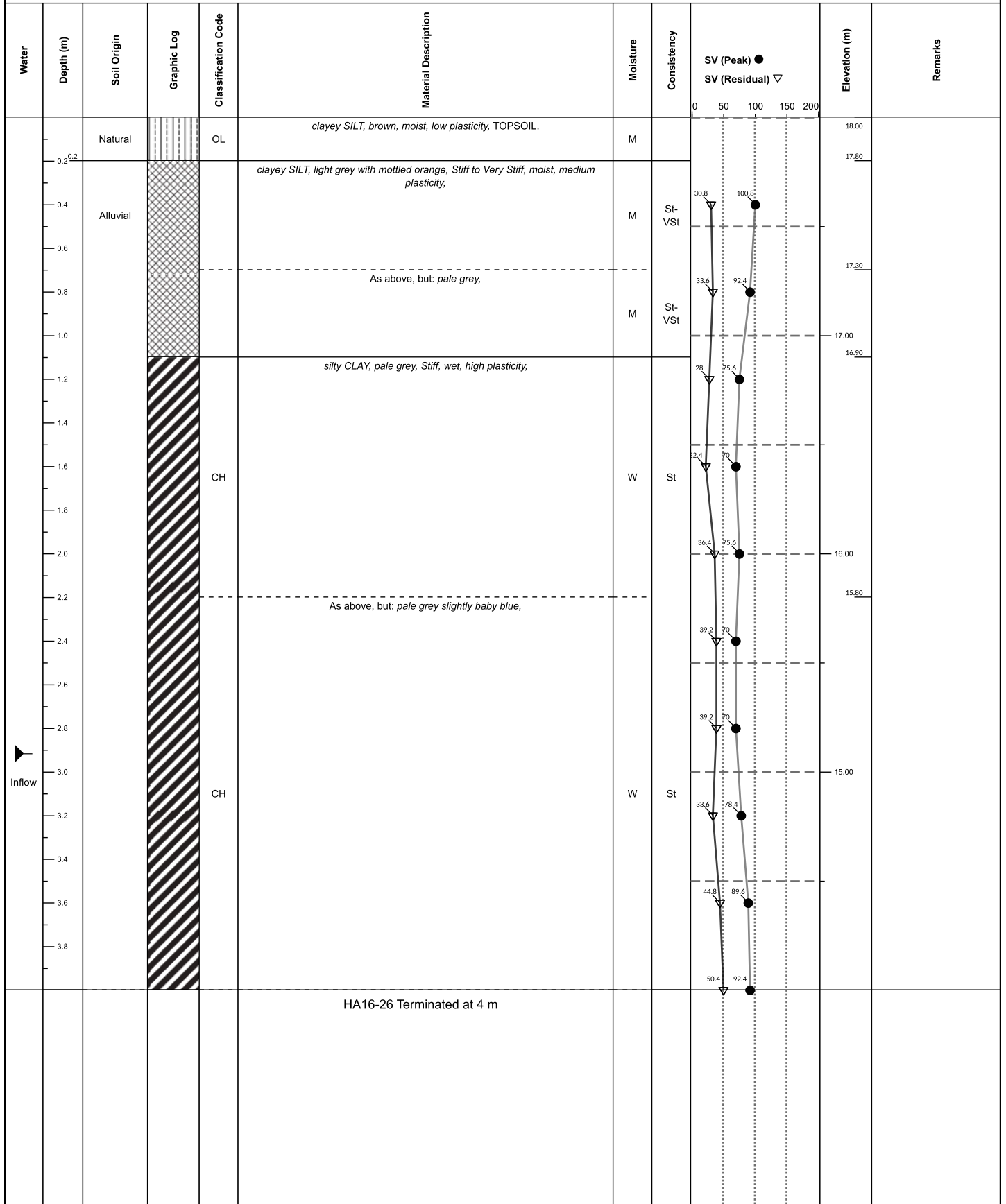
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Client : Fulton Hogan Land Development Ltd

Project : Milldale North

Location : 522 Wainui Road, Wainui 0992, New Zealand

Loc Comment :



Coordinate System : EPSG:2193

Easting (m) : 1746918.75

Northing (m) : 5948570.30

Ground Elevation : 18.00 (m)

Total Depth : 4 m BGL

Drill Rig : Hand Auger

Driller Supplier : Gibb Ritchie Geotechnical Engineers Ltd

Logged By : Ilhan Serif

Reviewed By :

Date : 25/05/2026

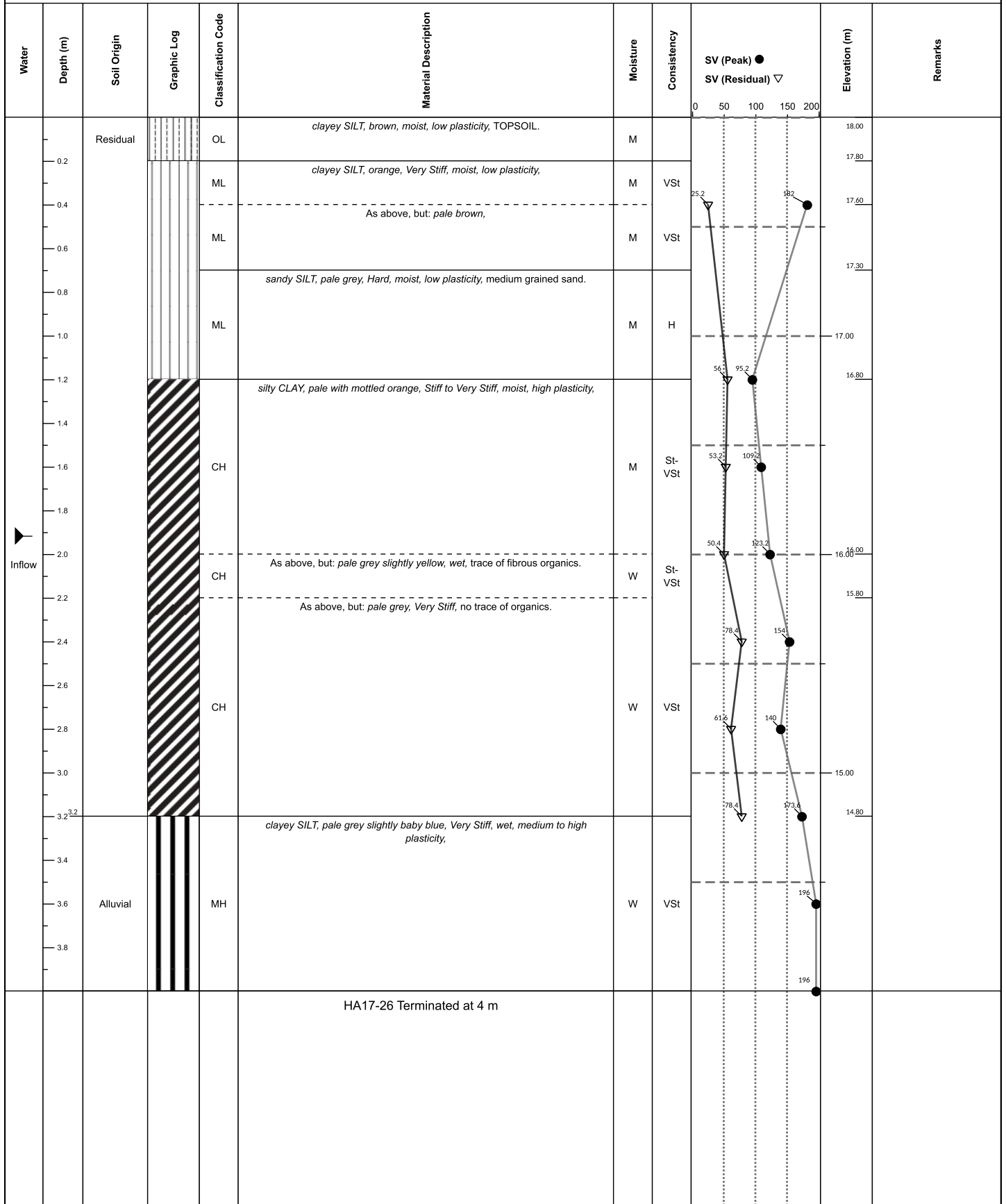
Job Number : AK26-027

Client : Fulton Hogan Land Development Ltd

Project : Milldale North

Location : 522 Wainui Road, Wainui 0992, New Zealand

Loc Comment :



Coordinate System : EPSG:2193

Easting (m) : 1746973.43

Northing (m) : 5948512.99

Ground Elevation : 15.48 (m)

Total Depth : 4 m BGL

Drill Rig : Hand Auger

Driller Supplier : Gibb Ritchie Geotechnical Engineers Ltd

Logged By : Ilhan Serif

Reviewed By :

Date : 27/05/2026

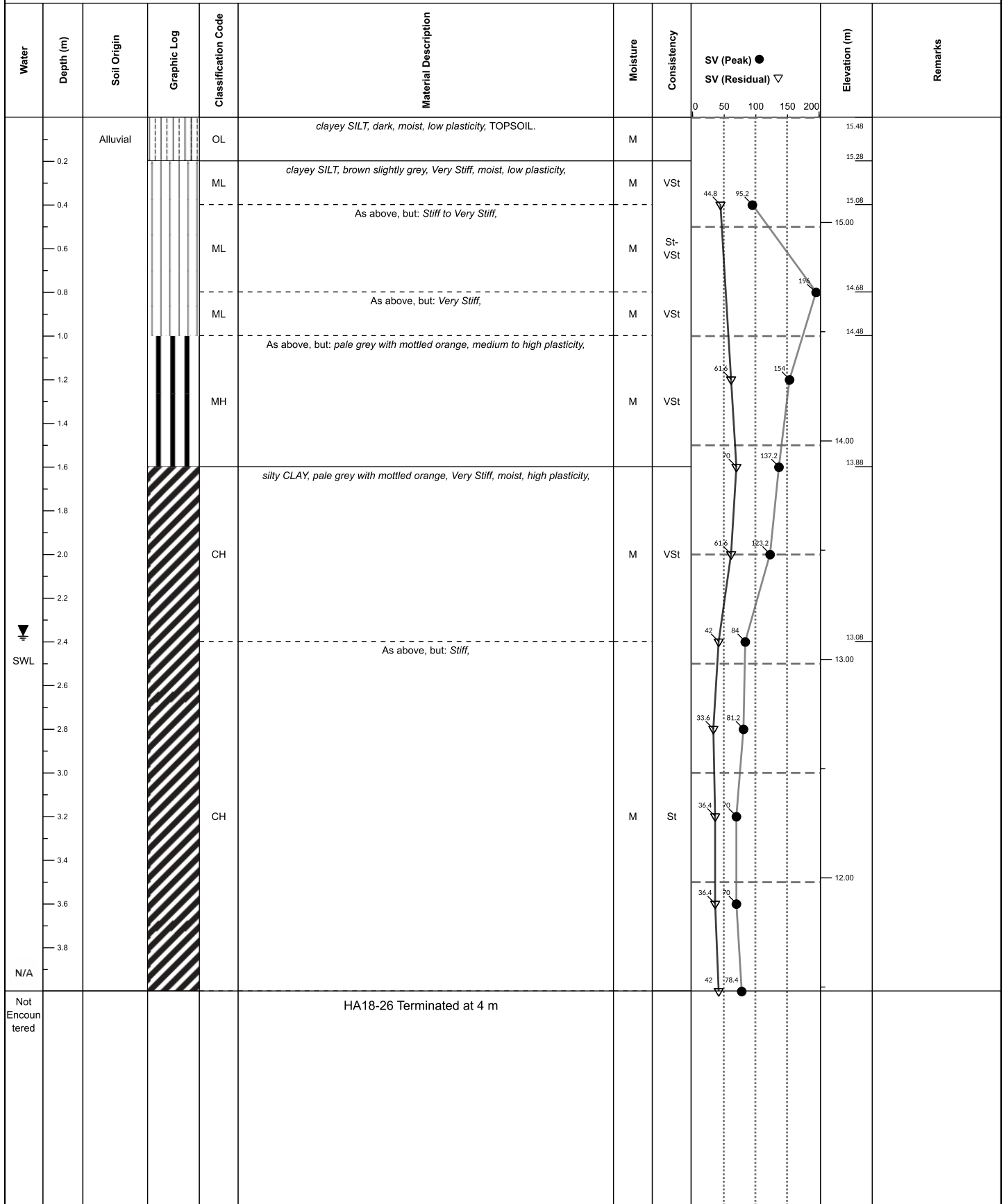
Job Number : AK26-027

Client : Fulton Hogan Land Development Ltd

Project : Milldale North

Location : 522 Wainui Road, Wainui 0992, New Zealand

Loc Comment :



Coordinate System : EPSG:2193

Easting (m) : 1746967.66

Northing (m) : 5948594.32

Ground Elevation : 17.00 (m)

Total Depth : 4 m BGL

Drill Rig : Hand Auger

Driller Supplier : Gibb Ritchie Geotechnical Engineers Ltd

Logged By : Ilhan Serif

Reviewed By :

Date : 27/05/2026

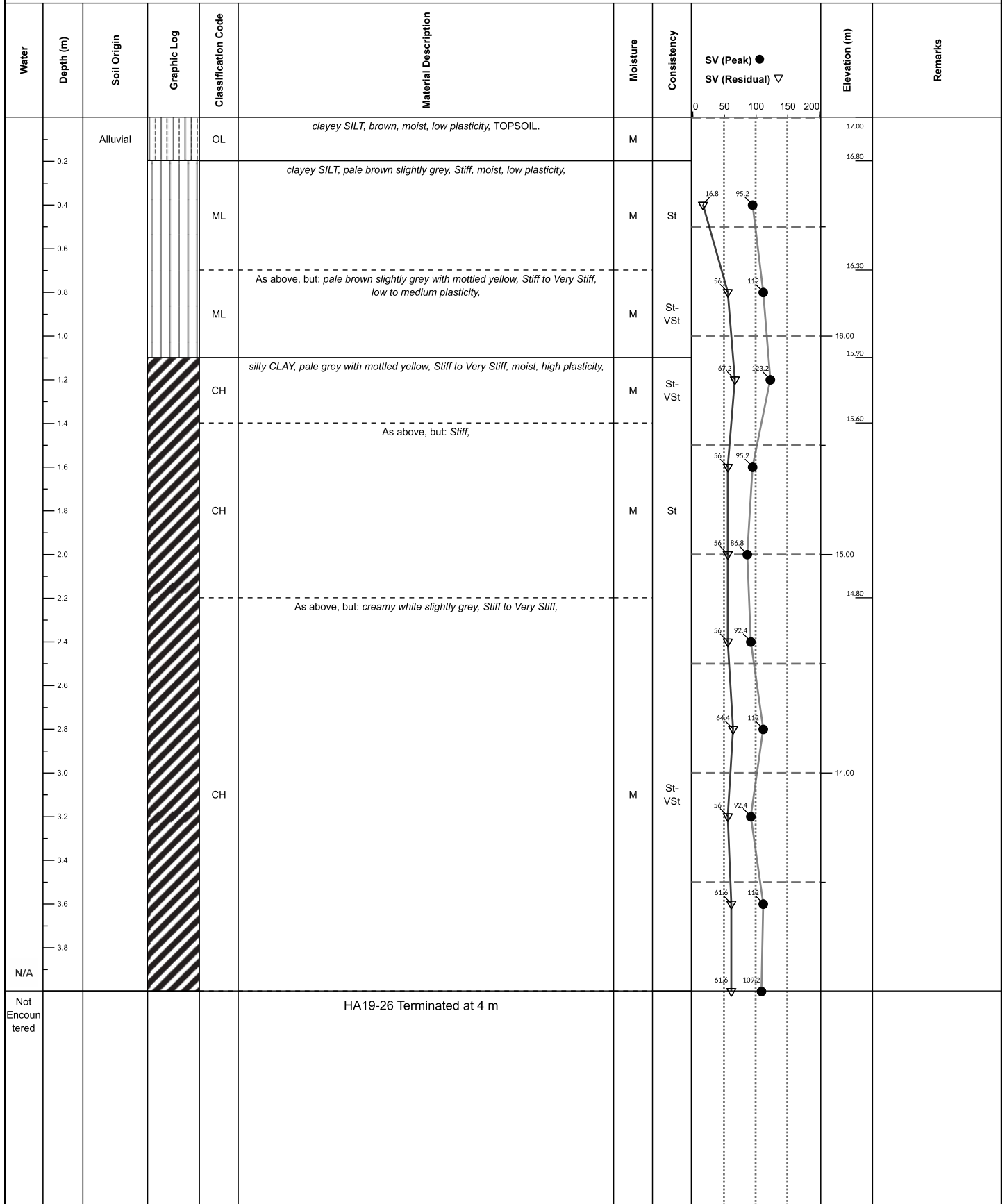
Job Number : AK26-027

Client : Fulton Hogan Land Development Ltd

Project : Milldale North

Location : 522 Wainui Road, Wainui 0992, New Zealand

Loc Comment :



Coordinate System : EPSG:2193

Easting (m) : 1747007.87

Northing (m) : 5948559.68

Ground Elevation : 15.41 (m)

Total Depth : 4 m BGL

Drill Rig : Hand Auger

Driller Supplier : Gibb Ritchie Geotechnical Engineers Ltd

Logged By : Ilhan Serif

Reviewed By :

Date : 27/05/2026

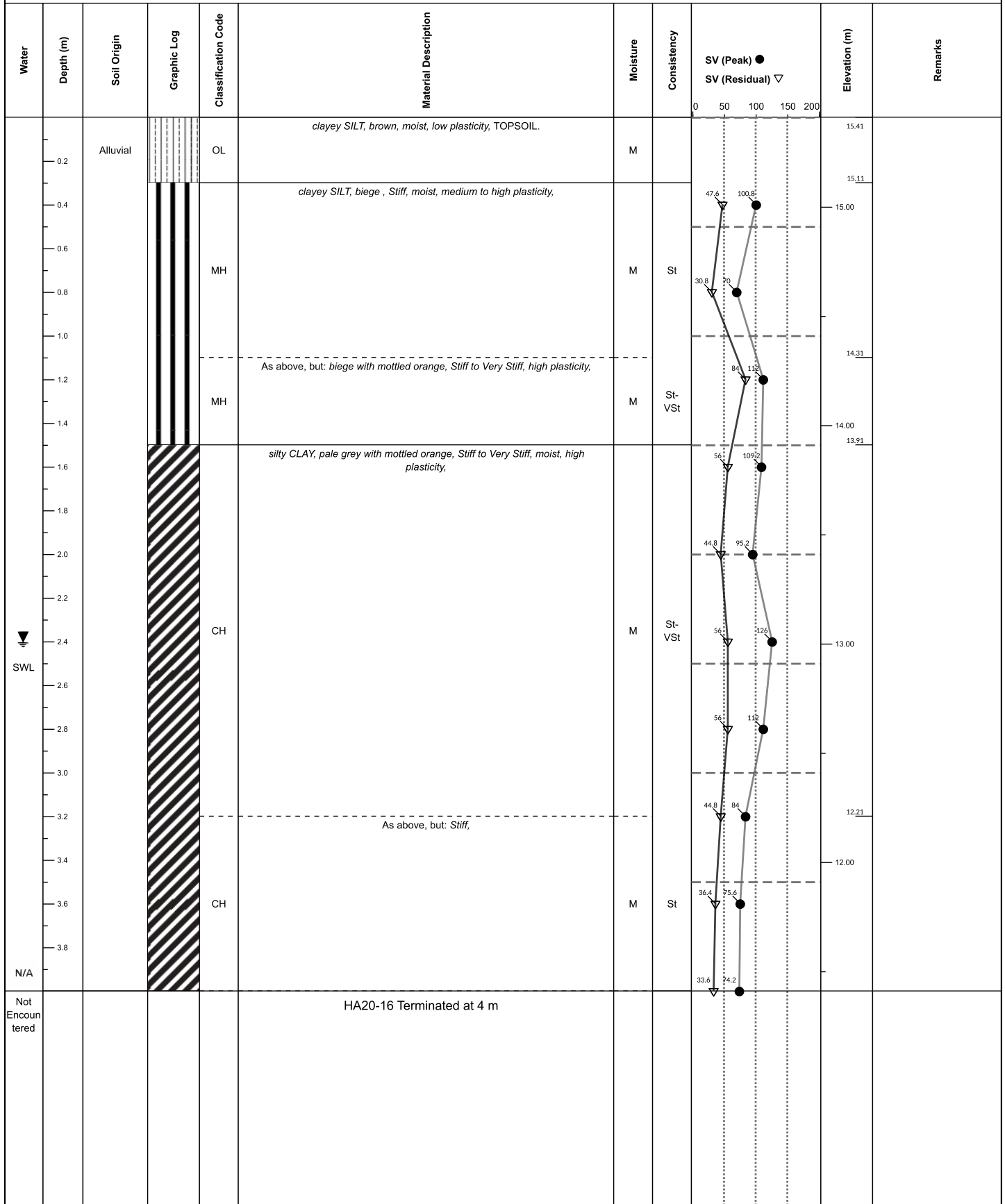
Job Number : AK26-027

Client : Fulton Hogan Land Development Ltd

Project : Milldale North

Location : 522 Wainui Road, Wainui 0992, New Zealand

Loc Comment :



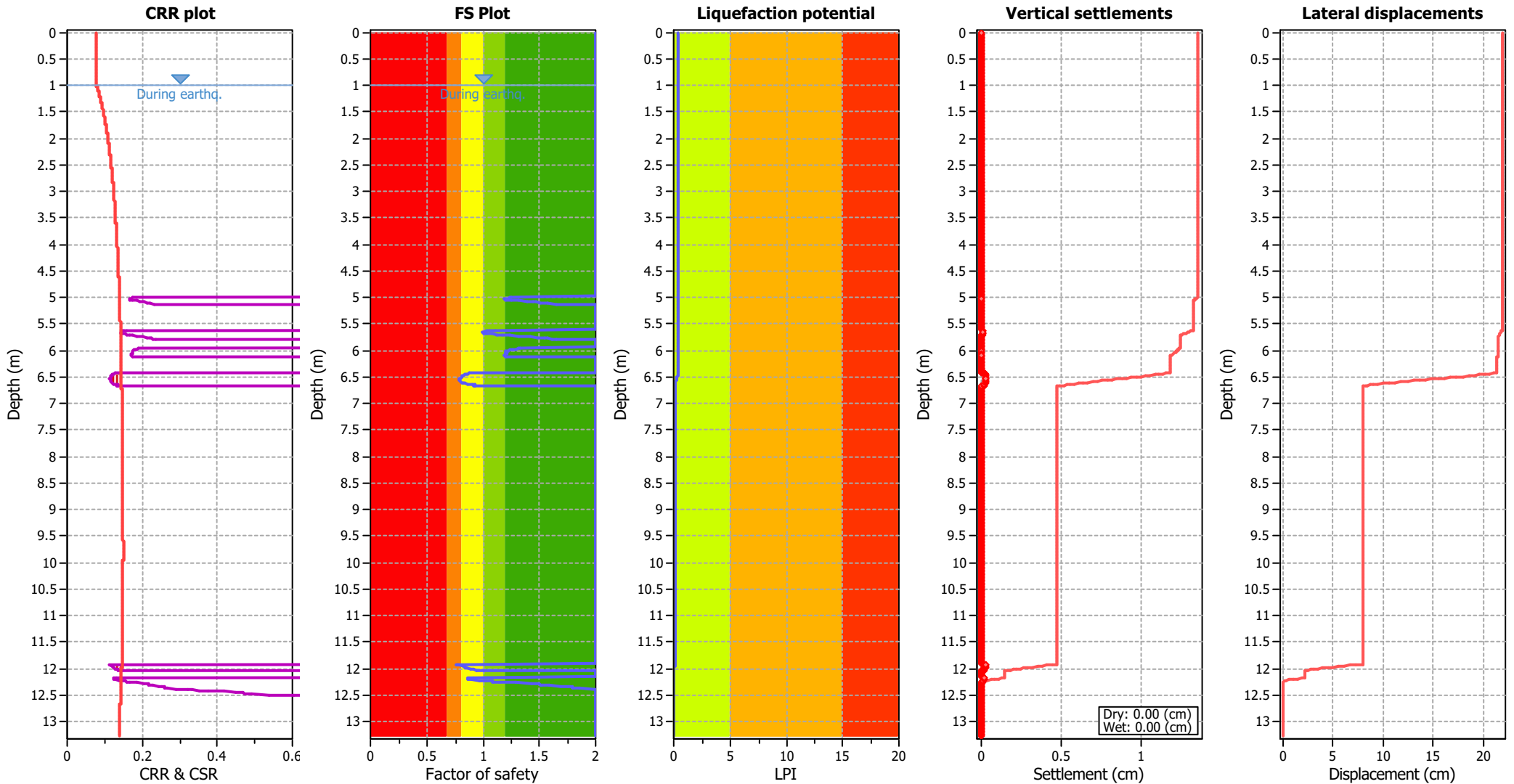
TF Topdrill Geotech		CONE PENETRATION TEST LOG Standard: CETANZ TG6					Topdrill Ref: TD4729 CPT NO.: CPT37-26 TEST NO.: 1	
PROJECT: Geotechnical Assessment		CLIENT: Gibb Ritchie Geotechnical Engineers		CONTRACTOR: TOPDRILL LTD		Start Date: 09/06/2026		
SITE LOCATION: Milldale North		CLIENT REF: AK26-027		OPERATOR: Jon Ellis		Start Time: 7:11 pm		
CO-ORDINATES: 1746863.08 mE, 5948599.06 mN (NZTM)		ENGINEER: Chris Ritchie		FIELD MANAGER: Greg K		Start Depth: 0.00 m		
Machine: Pagani, TG 63-150		Setup: None		ISO & TG6: None		Target Depth: 20 m		
Ballasted or Anchored: Anchored		Deviations:		Deviations:		Target Application Class: 2		
Maximum Push Capacity: 150kN						Acheived Application Class: 2		
Cone Type: TE2		Cone Class: 2		Compression or Subtraction: Subtraction		Filter Location: u2		
Manufacturer: Pagani		Nominal Size: 10cm²		Net Area Ratio: 0.7955cm²		Filter Type: POROUS		
Serial No: 000633		Calibration Ref: z069/26		Friction Reducer: ☒		Filter Saturation: FUNNEL		
						Temperature: -		
						Temperature Conditioning: No conditioning - already close to ground temperature		

TF Topdrill Geotech		CONE PENETRATION TEST LOG Standard: CETANZ TG6					Topdrill Ref: TD4729 CPT NO.: CPT38-26 TEST NO.: 1						
PROJECT: Geotechnical Assessment		CLIENT: Gibb Ritchie Geotechnical Engineers		CONTRACTOR: TOPDRILL LTD		Start Date: 09/06/2026							
SITE LOCATION: Milldale North		CLIENT REF: AK26-027		OPERATOR: Jon Ellis		Start Time: 5:52 pm							
CO-ORDINATES: 1746902.08 mE, 5948641.52 mN (NZTM)		ENGINEER: Chris Ritchie		FIELD MANAGER: Greg K		Start Depth: 0.00 m							
Machine: Pagani, TG 63-150		Setup None		ISO & TG6 None		Target Depth: 20 m							
Ballasted or Anchored: Anchored		Deviations:		Deviations:		Target Application Class: 2							
Maximum Push Capacity: 150kN						Acheived Application Class: 2							
Cone Type: TE2		Cone Class: 2		Compression or Subtraction: Subtraction		Temperature: -							
Manufacturer: Pagani		Nominal Size: 10cm²		Net Area Ratio: 0.7955cm²		Temperature Conditioning:							
Serial No: 000633		Calibration Ref: z069/26		Friction Reducer: ☒		No conditioning - already close to ground temperature							
				Filter Location: u2									
				Filter Type: POROUS									
				Filter Saturation: FUNNEL									
Depth (m)	Tip Resistance (MPa)		Pore Pressure (kPa)	Friction Ratio (%)	SBT	SBT Description (filtered)	Depth (m)	Vertical Stress (kPa) Total Effective	Unit Weight (kN/m3)	Friction Angle (°)	Dr (%)		N ₆₀
	Sleeve Friction (kPa)										Su (kPa)		
1						Clays: clay to silty clay	1						
2							2						
3						Clays: clay to silty clay	3						
4							4						
5						Silt mixtures: clayey silt & silty clay	5						
6						Clays: clay to silty clay	6						
7						Clays: clay to silty clay	7						
8							8						
9						Silt mixtures: clayey silt & silty clay	9						
10						Clays: clay to silty clay	10						
11						Silt mixtures: clayey silt & silty clay	11						
12							12						
13						Silt mixtures: clayey silt & silty clay	13						
14						Silt mixtures: clayey silt & silty clay	14						
15							15						
16						Silt mixtures: clayey silt & silty clay	16						
EOH: 16.54m													
GWL - Predrill - Backfill			Zero Readings			Soil Behaviour Type (SBT) - Robertson et al. 1986			NOTES & LIMITATIONS			REMARKS:	
Measured Water Level:			Tip Resistance Initial: 0.0210 MPa			0 Undefined 1 Sensitive fine-grained 2 Clay - organic soil 3 Clays: clay to silty clay 4 Silt mixtures: clayey silt & silty clay 5 Sand mixtures: silty sand to sandy silt 6 Sands: clean sands to silty sands 7 Dense sand to gravelly sand 8 Stiff sand to clayey sand 9 Stiff fine-grained			Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P. K. Robertson and K.L. Cabal (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 4th Edition. The interpretations are presented only as a guide for geotechnical use, and should be carefully reviewed by the user. Geroc Solutions Ltd do not warranty the correctness or the applicability of any of the geotechnical soil and design parameters shown and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used to derive data shown in this report.			TERMINATION: High Total Load	
Groundwater Method: Dipmeter (dry 2.10m)			Tip Resistance Final: 0.0210 MPa										
Pore Pressure Response: Good			Local Friction Initial: 0.2383 kPa										
Pre-drill Depth: N/A			Local Friction Final: 0.2385 kPa										
Pre-drill Reason: N/A			Pore Pressure Initial: 3.0325 kPa										
Pre-drill Backfill: N/A			Pore Pressure Final: 3.0332 kPa										
Backfill Method: N/A													

APPENDIX C

CLIQ Outputs

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	5.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

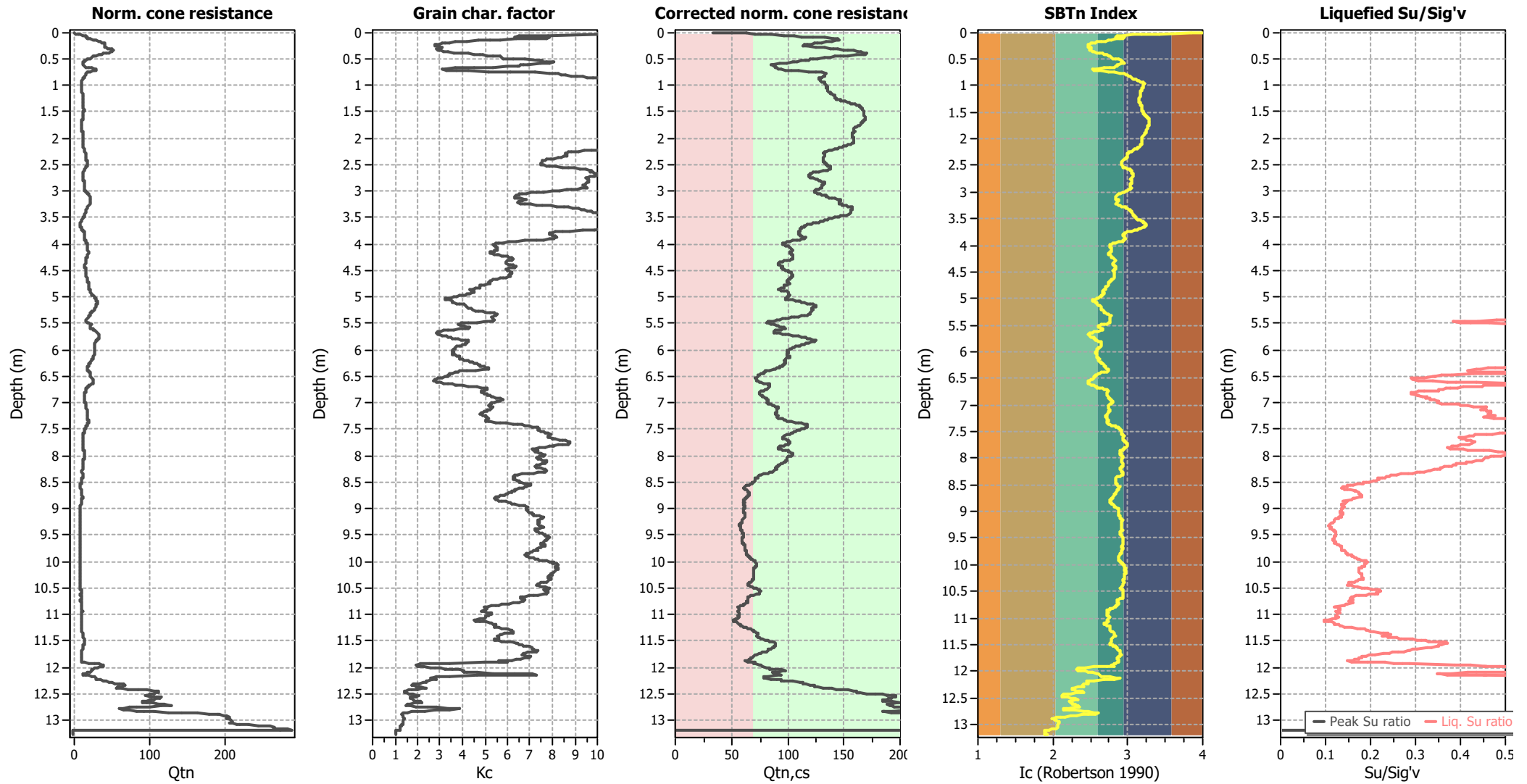
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

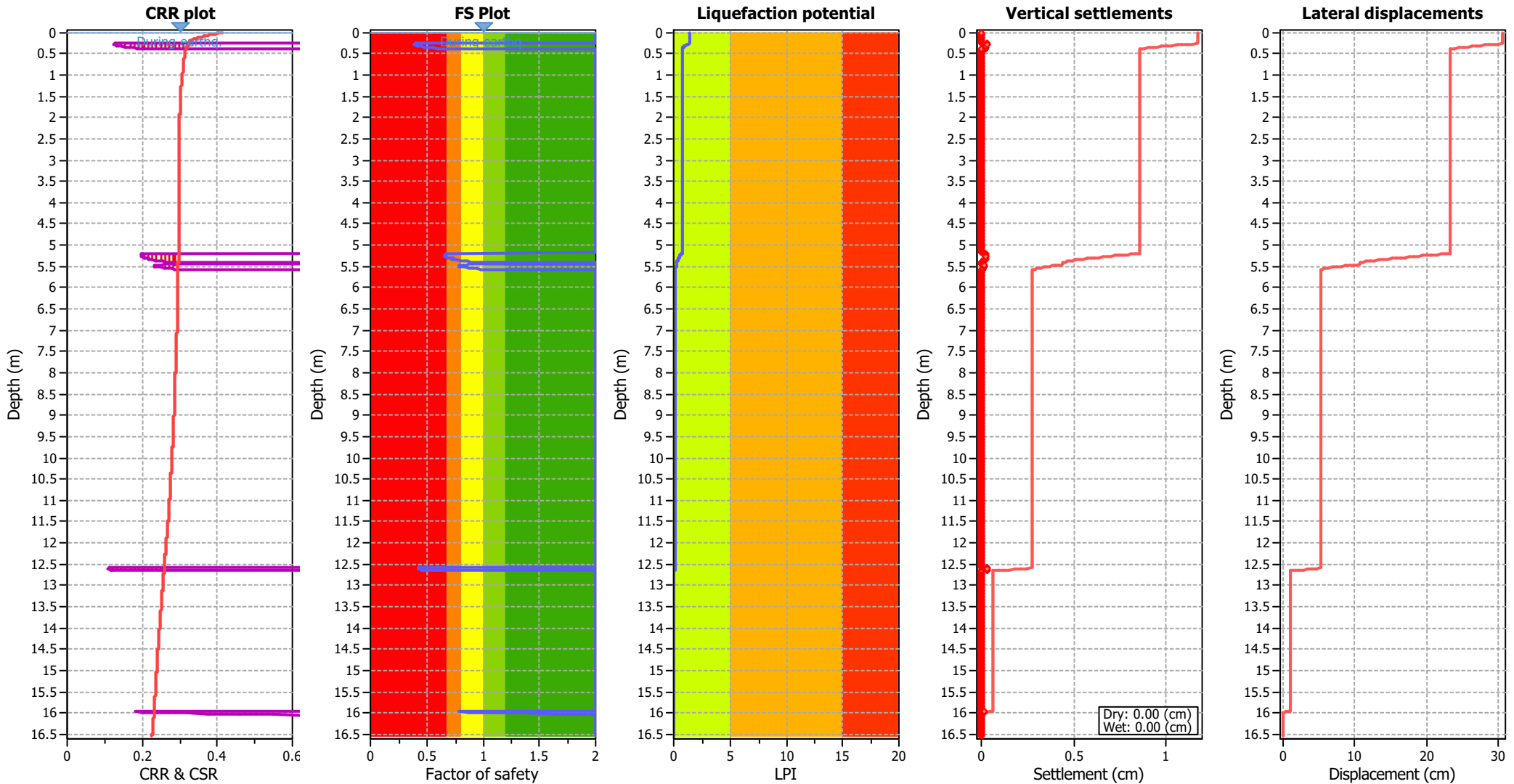
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	5.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.24	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	0.00 m	Fill height:	N/A	Limit depth:	N/A

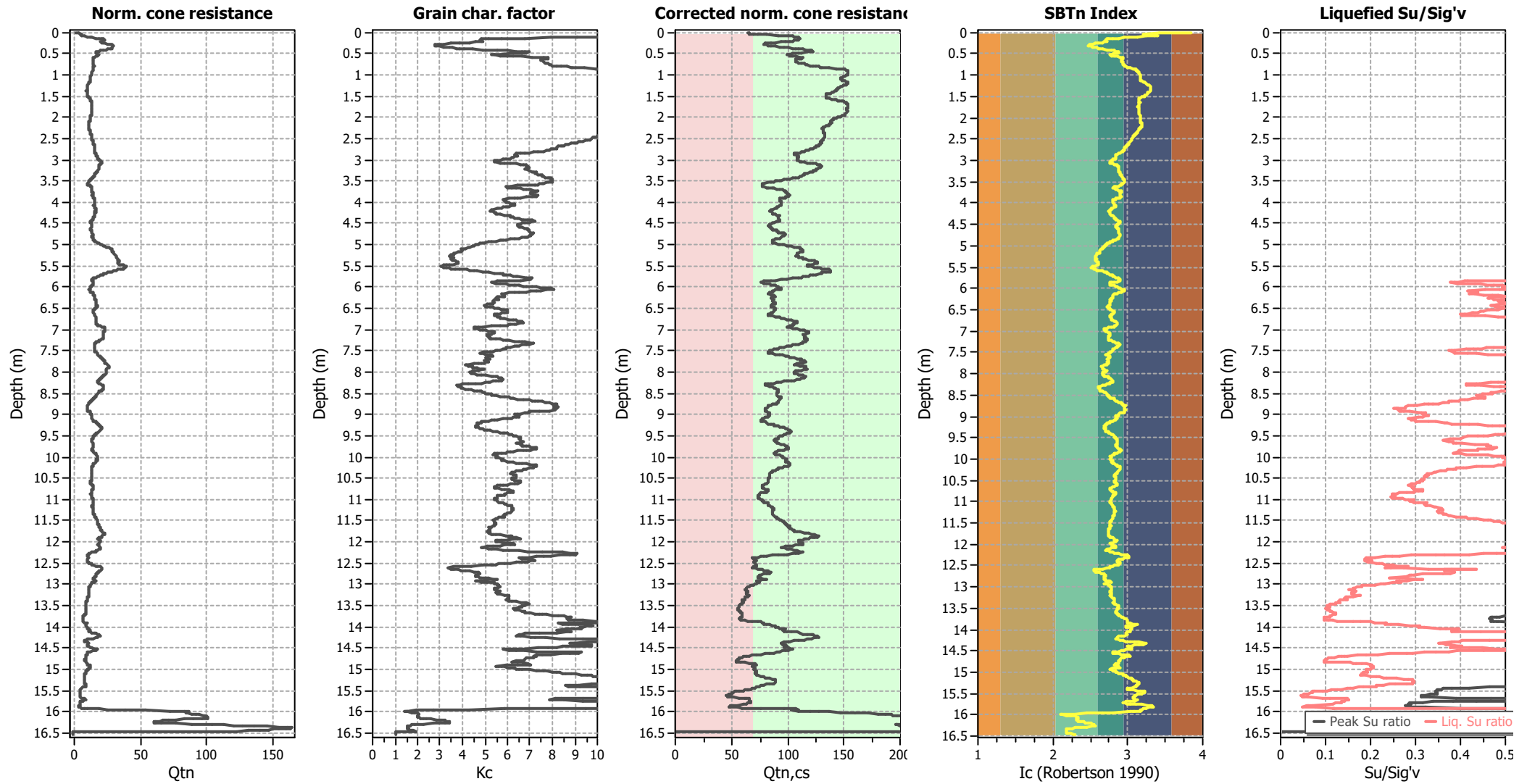
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

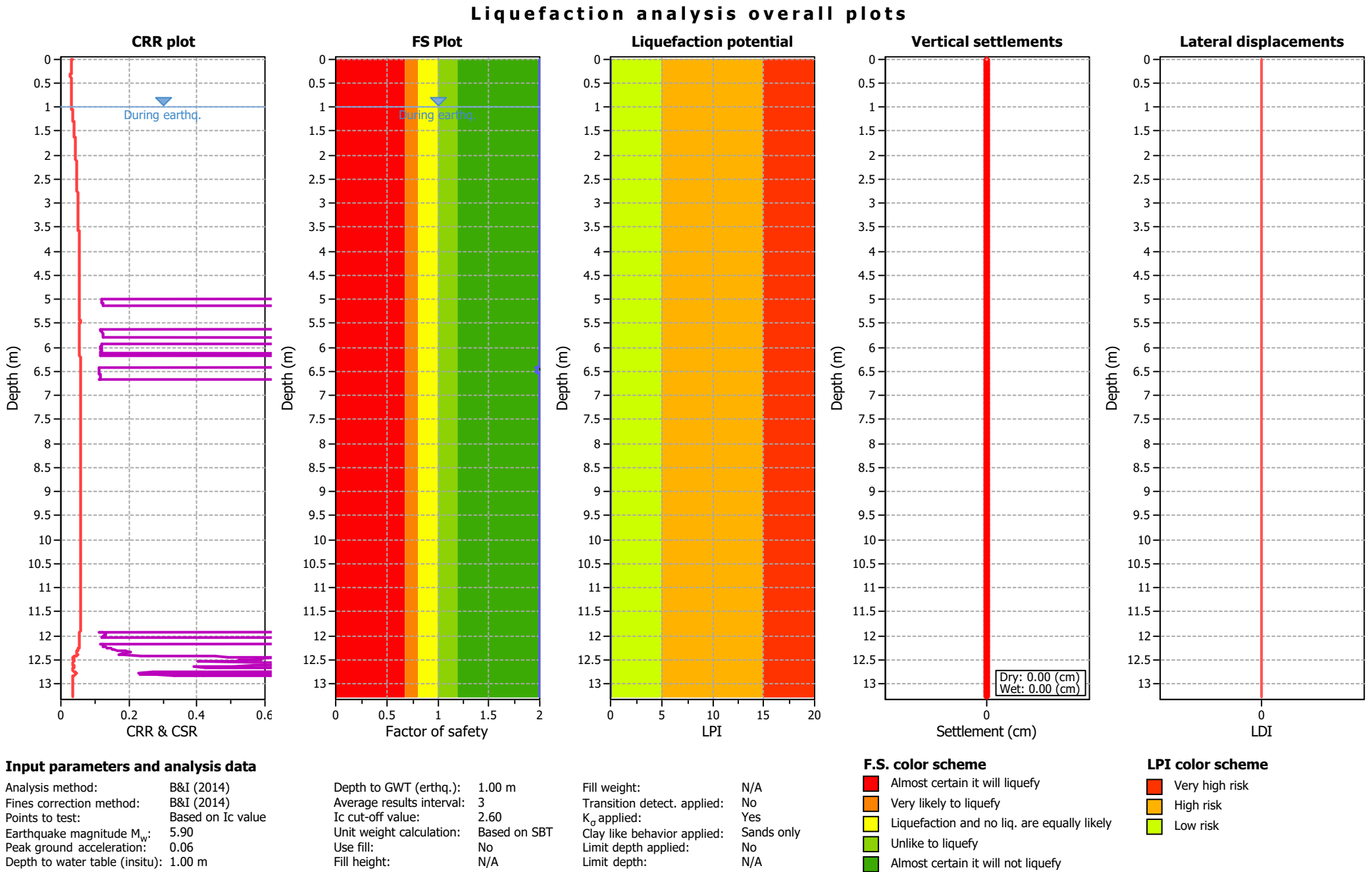
- Very high risk
- High risk
- Low risk

Check for strength loss plots (Robertson (2010))

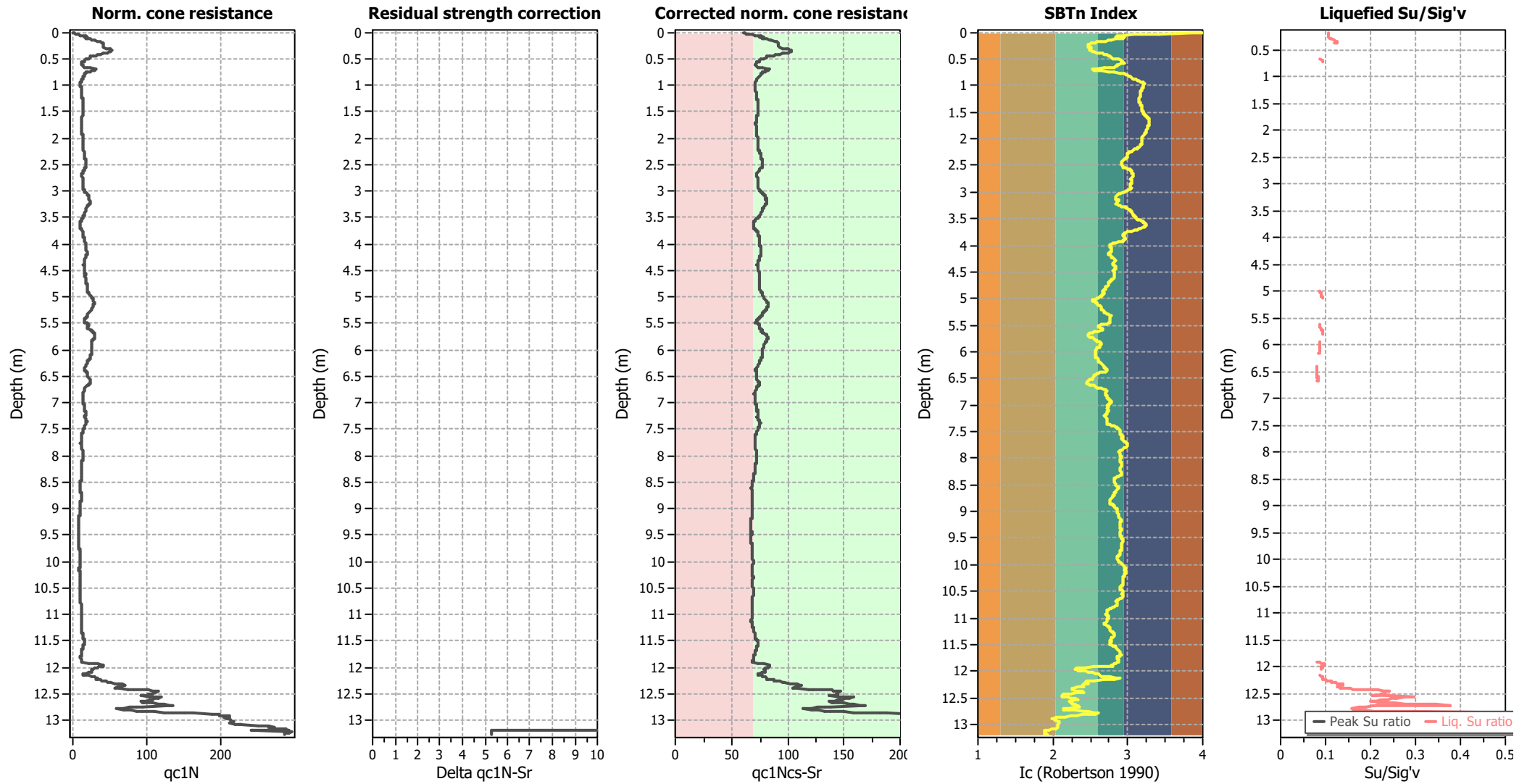


Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	0.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.24	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	0.00 m	Fill height:	N/A	Limit depth:	N/A



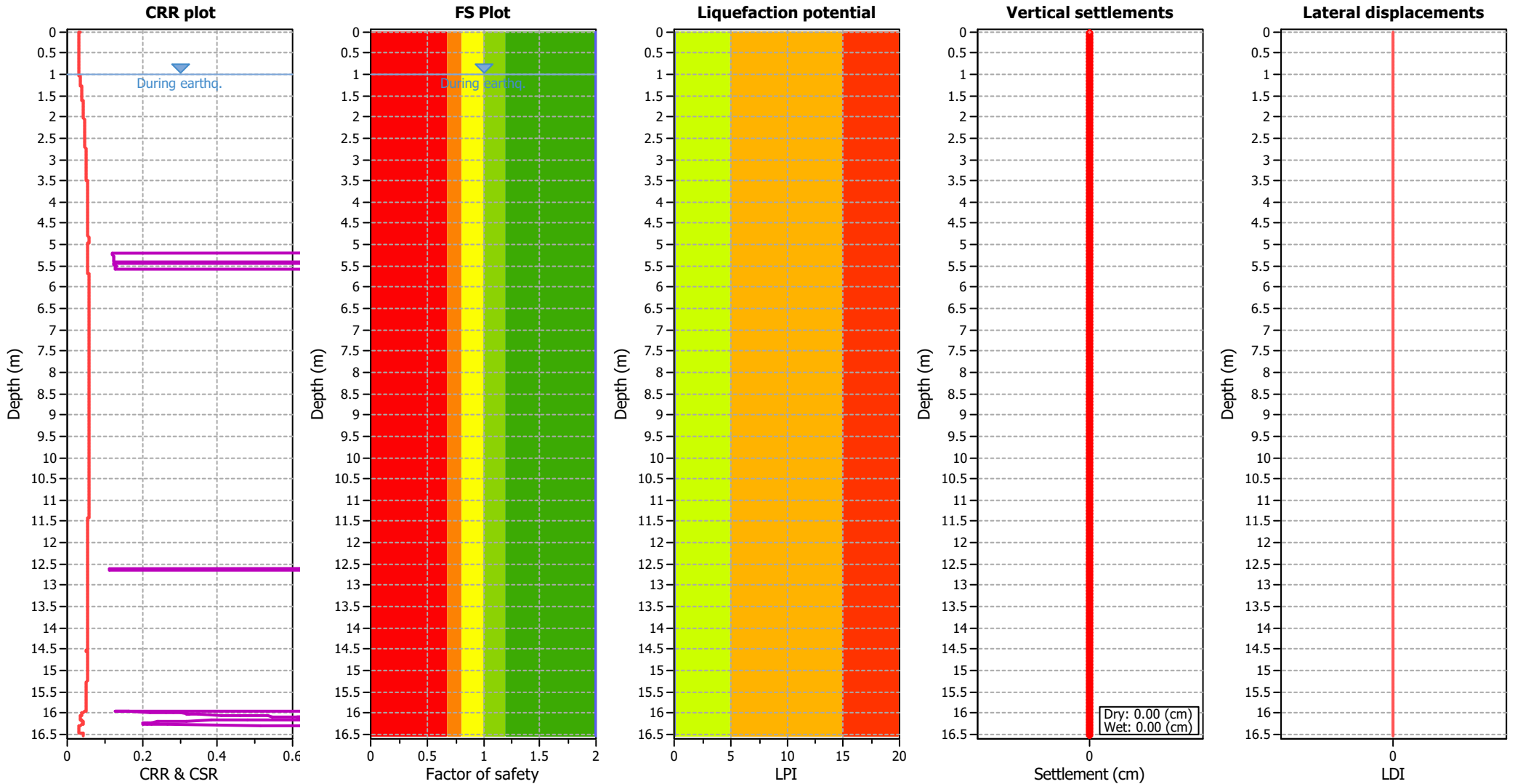
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.06	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.06	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

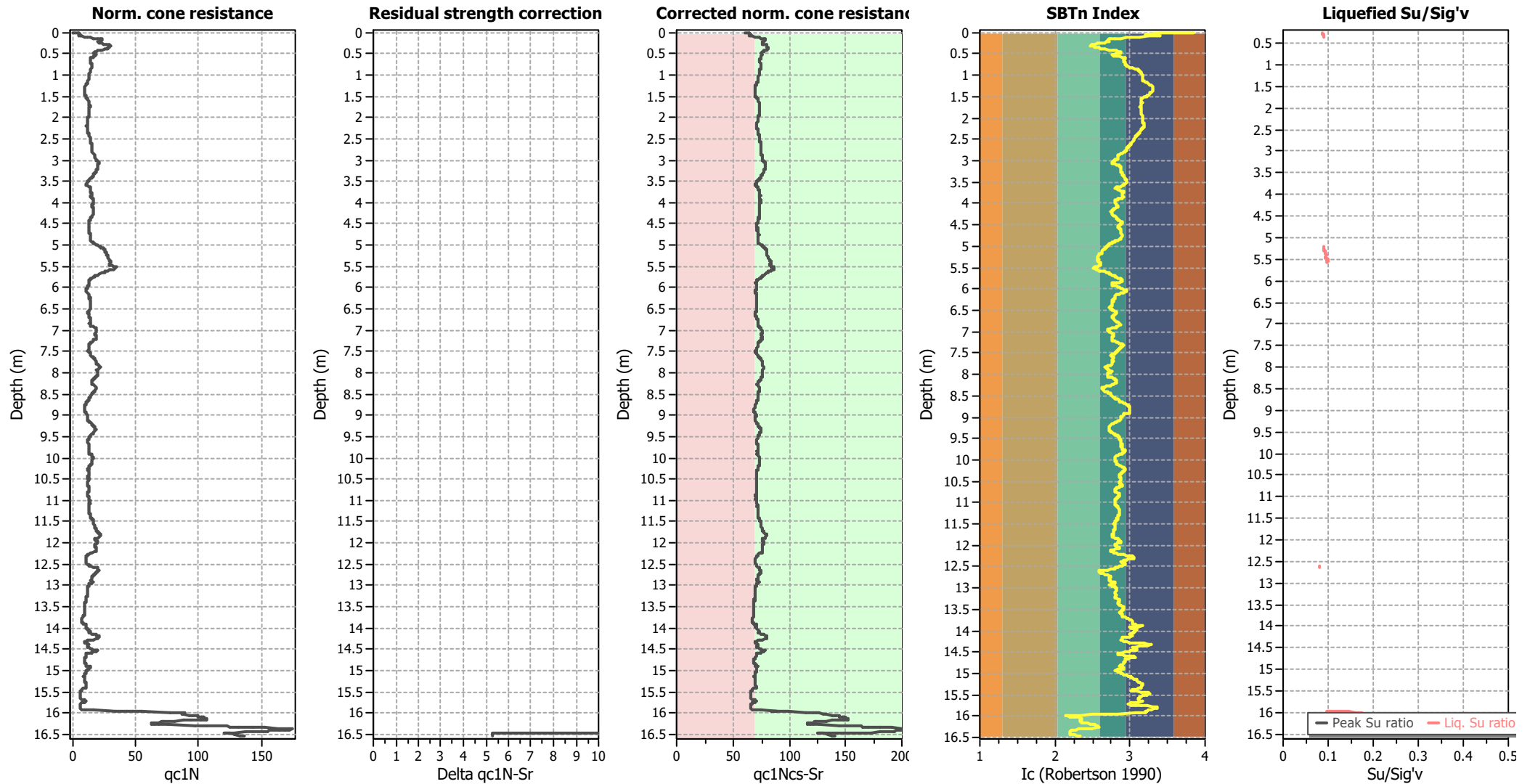
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Check for strength loss plots (Idriss & Boulanger (2008))

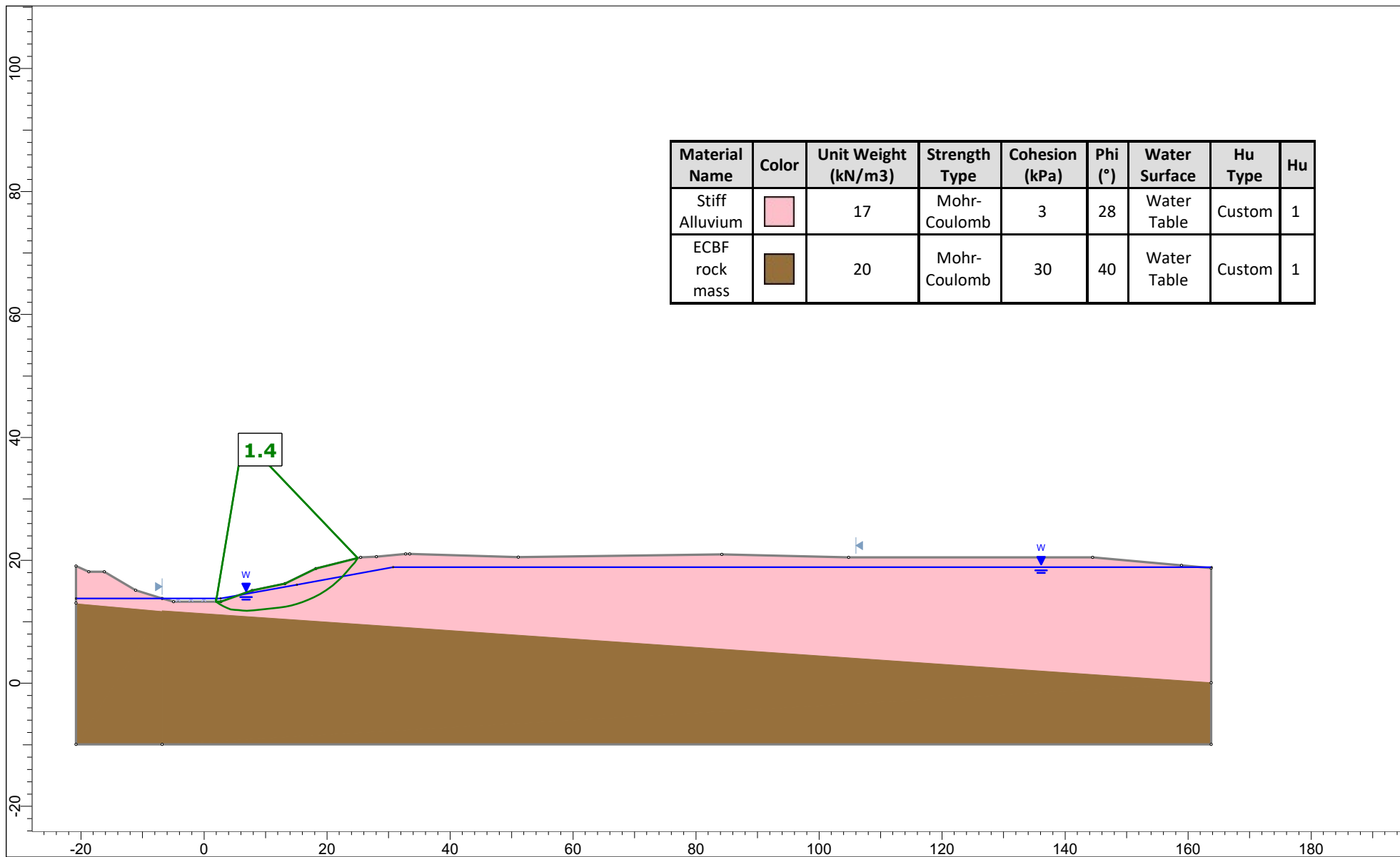


Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on I_c value	I_c cut-off value:	2.60	K_0 applied:	Yes
Earthquake magnitude M_w :	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.06	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

APPENDIX D

Slope Stability Outputs

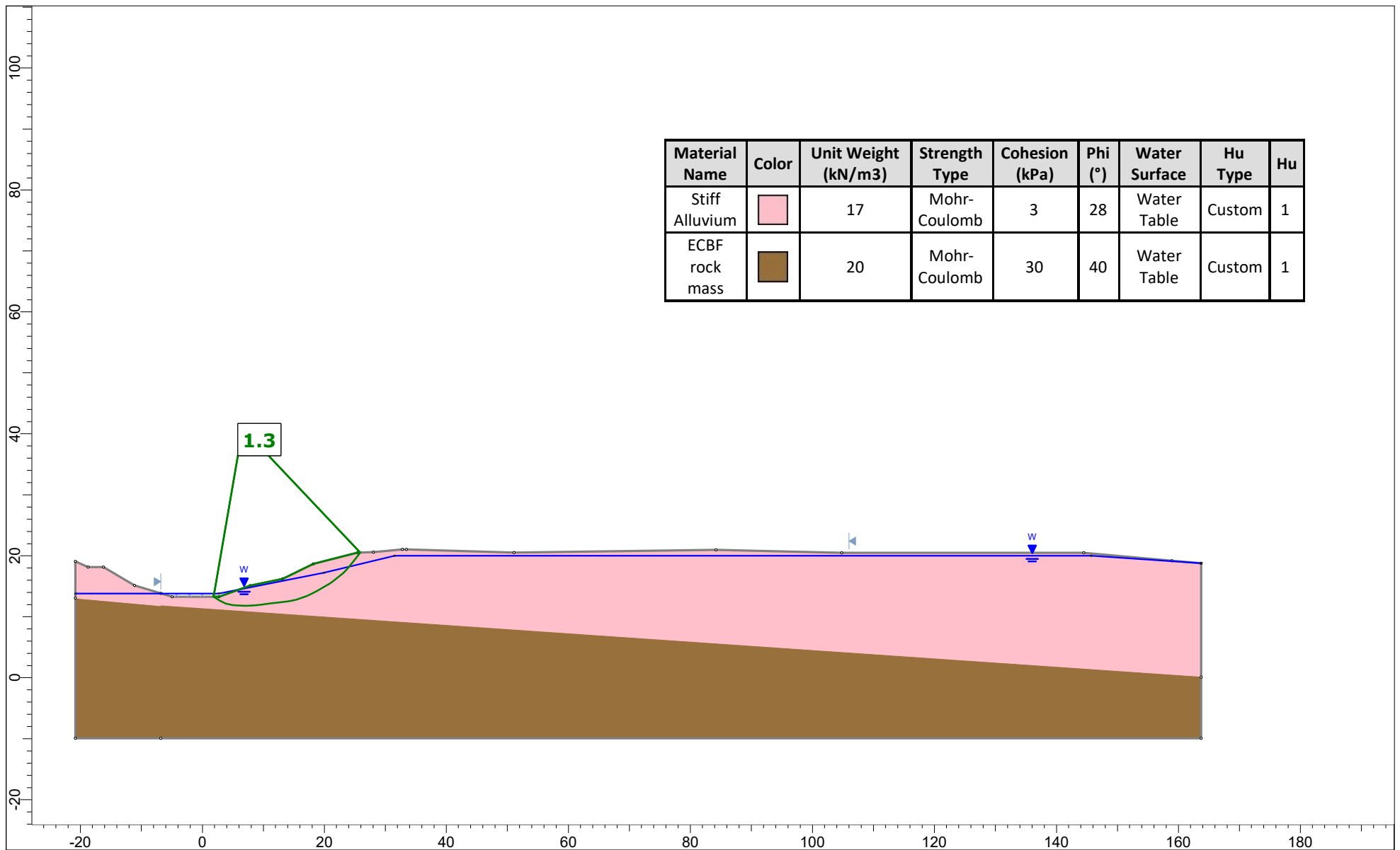


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type	Hu
Stiff Alluvium		17	Mohr-Coulomb	3	28	Water Table	Custom	1
ECBF rock mass		20	Mohr-Coulomb	30	40	Water Table	Custom	1



SLIDEINTERPRET 9.038

Project		Milldale Stages 14-17	
Group	Section XS N - Existing	Scenario	NGW
ST	CR	Company	Gibb Ritchie Geotechnical Engineers
Date	30/07/2026 2:01:46 pm	File Name	XS N.slm

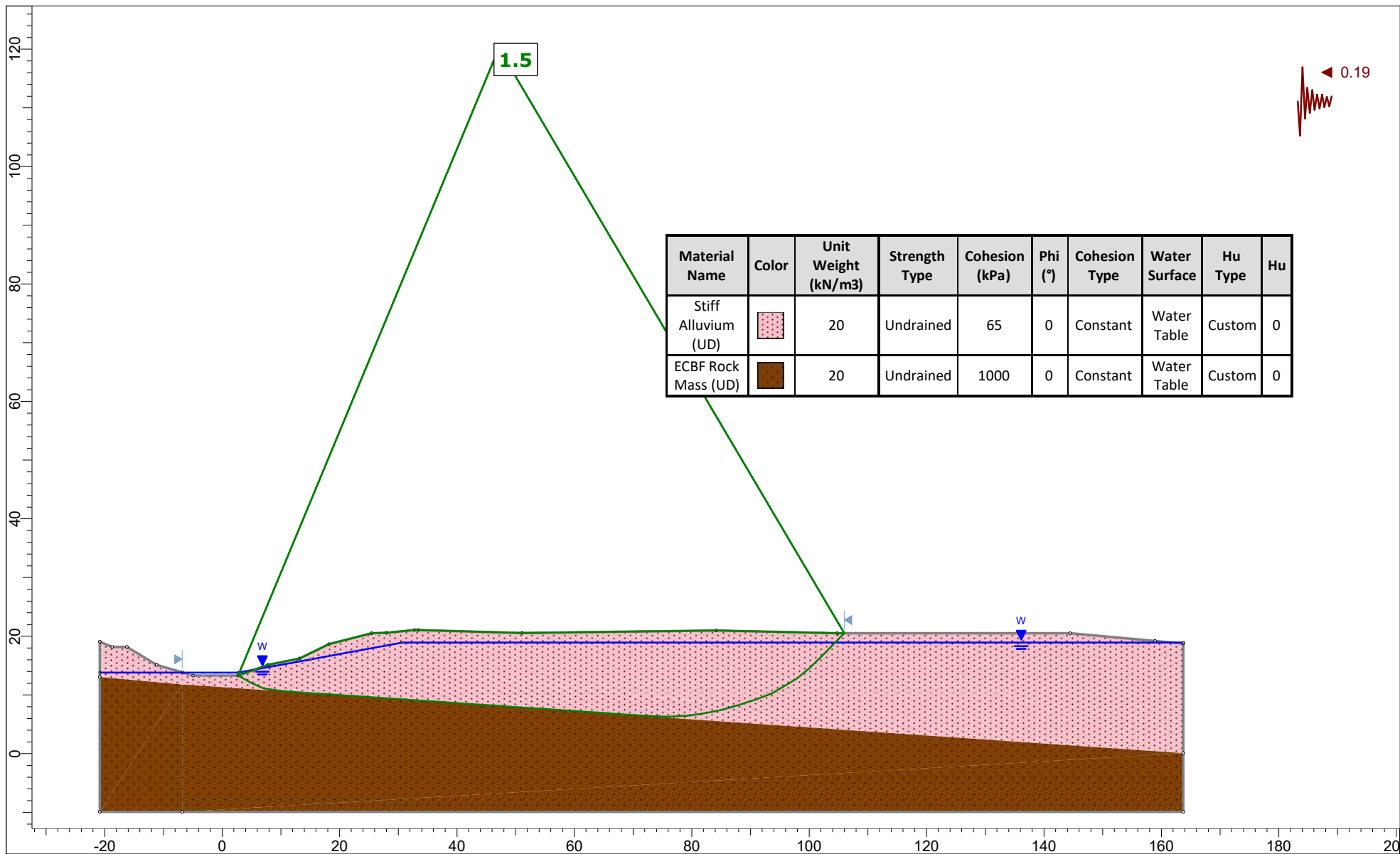


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type	Hu
Stiff Alluvium		17	Mohr-Coulomb	3	28	Water Table	Custom	1
ECBF rock mass		20	Mohr-Coulomb	30	40	Water Table	Custom	1



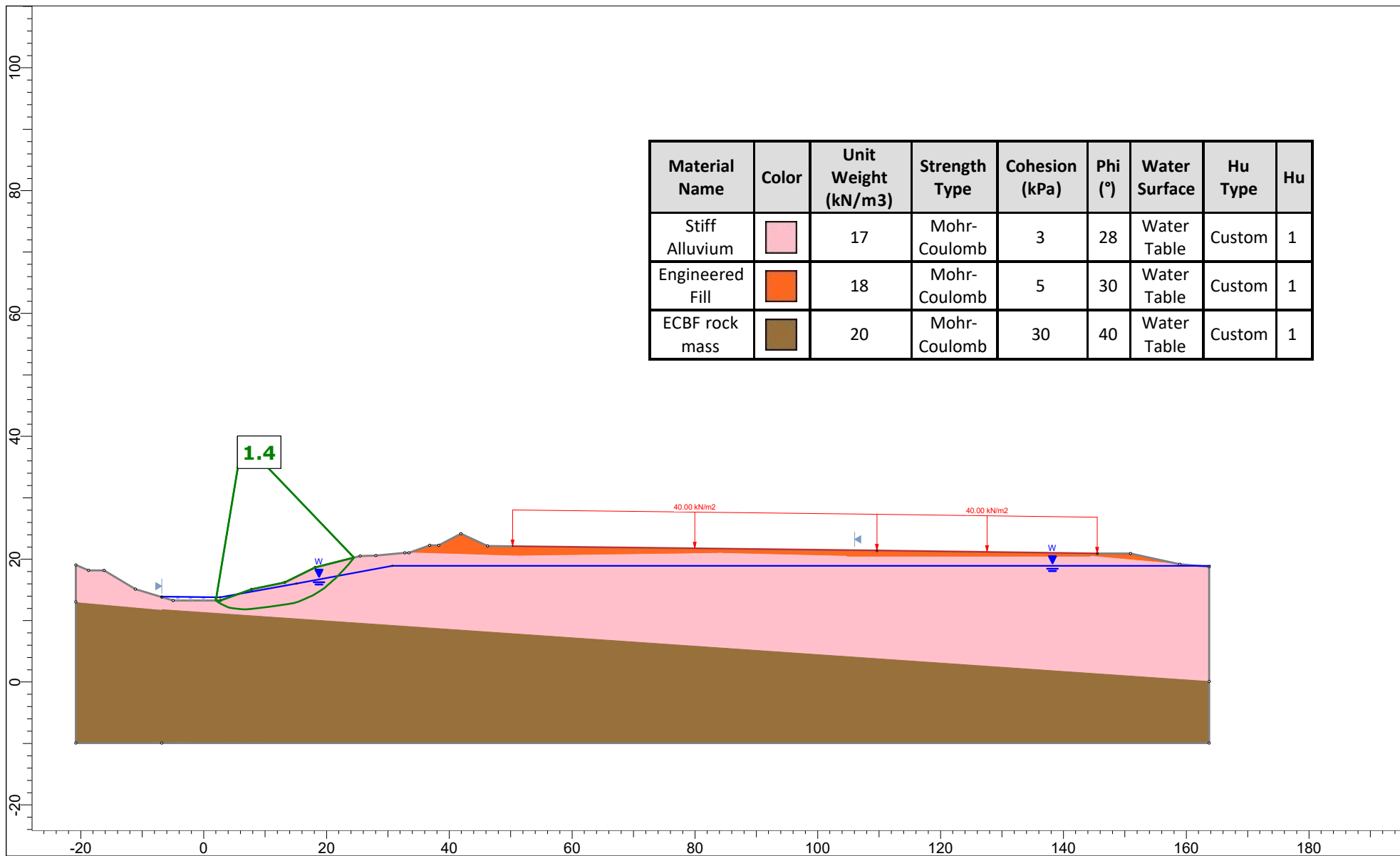
SLIDEINTERPRET 9.038

Project	Milldale Stages 14-17		
Group	Section XS N - Existing	Scenario	HGW
ST	CR	Company	Gibb Ritchie Geotechnical Engineers
Date	30/07/2026 2:01:46 pm	File Name	XS N.slm



SLIDEINTERPRET 9.038

Project		Milldale Stages 14-17	
Group	Section XS N - Existing	Scenario	Seismic
ST	CR	Company	Gibb Ritchie Geotechnical Engineers
Date	30/07/2026 2:01:47 pm	File Name	XS N.slm

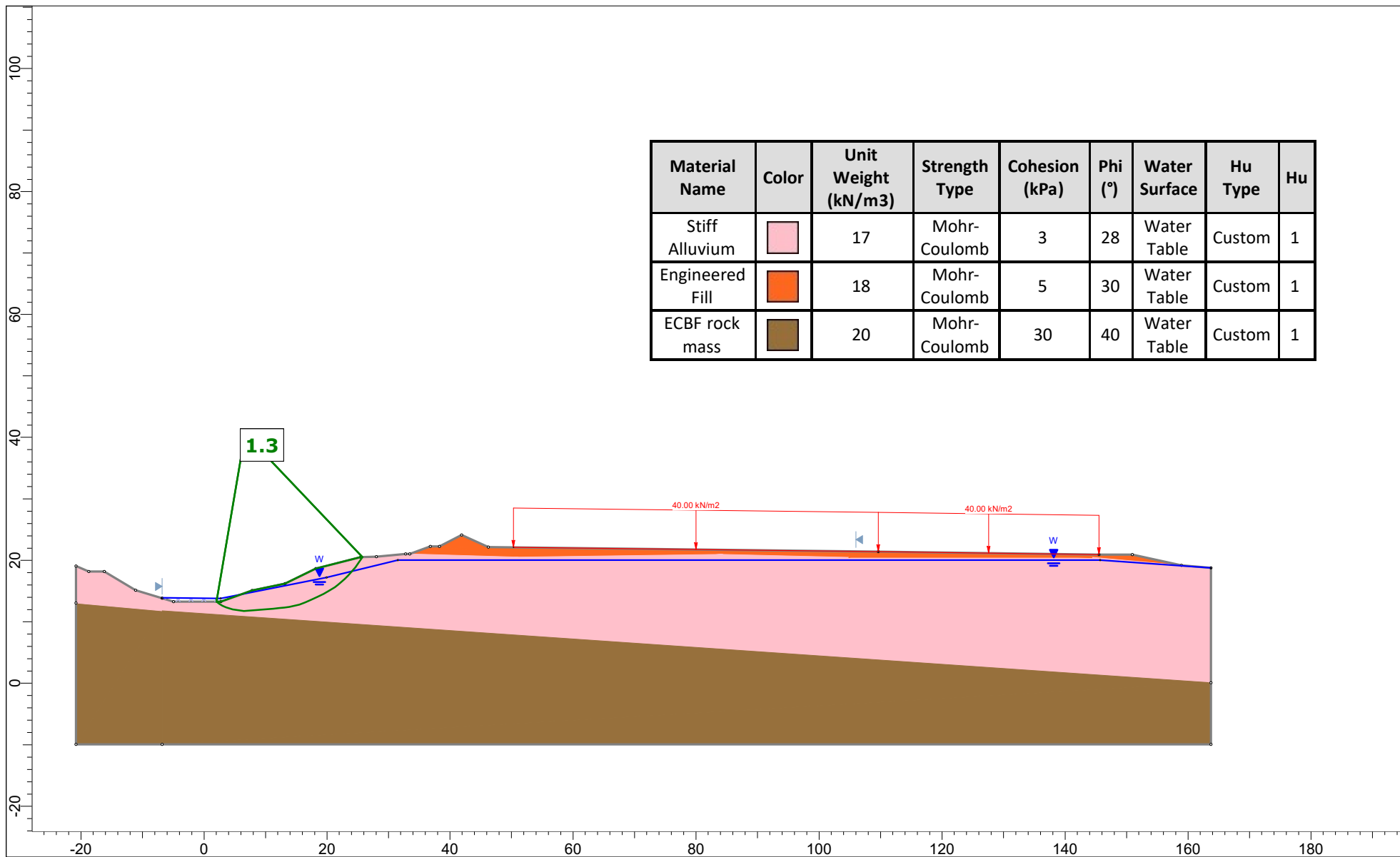


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type	Hu
Stiff Alluvium		17	Mohr-Coulomb	3	28	Water Table	Custom	1
Engineered Fill		18	Mohr-Coulomb	5	30	Water Table	Custom	1
ECBF rock mass		20	Mohr-Coulomb	30	40	Water Table	Custom	1



SLIDEINTERPRET 9.038

Project		Milldale Stages 14-17	
Group	Section XS N - Proposed	Scenario	NGW
ST	CR	Company	Gibb Ritchie Geotechnical Engineers
Date	30/07/2026 2:01:45 pm	File Name	XS N.slm

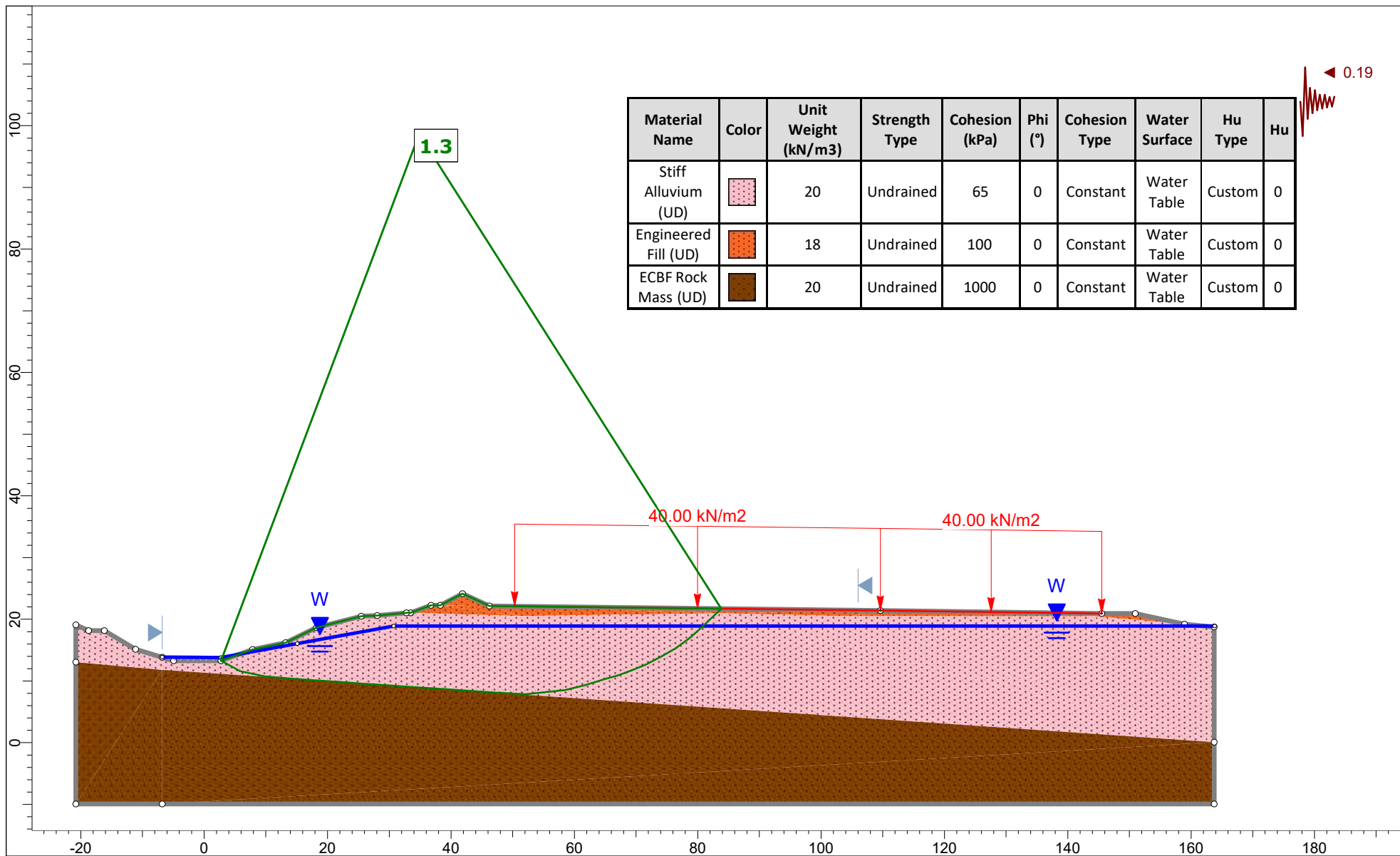


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type	Hu
Stiff Alluvium		17	Mohr-Coulomb	3	28	Water Table	Custom	1
Engineered Fill		18	Mohr-Coulomb	5	30	Water Table	Custom	1
ECBF rock mass		20	Mohr-Coulomb	30	40	Water Table	Custom	1



SLIDEINTERPRET 9.038

Project	Milldale Stages 14-17		
Group	Section XS N - Proposed	Scenario	HGW
ST	CR	Company	Gibb Ritchie Geotechnical Engineers
Date	30/07/2026 2:01:45 pm	File Name	XS N.slm

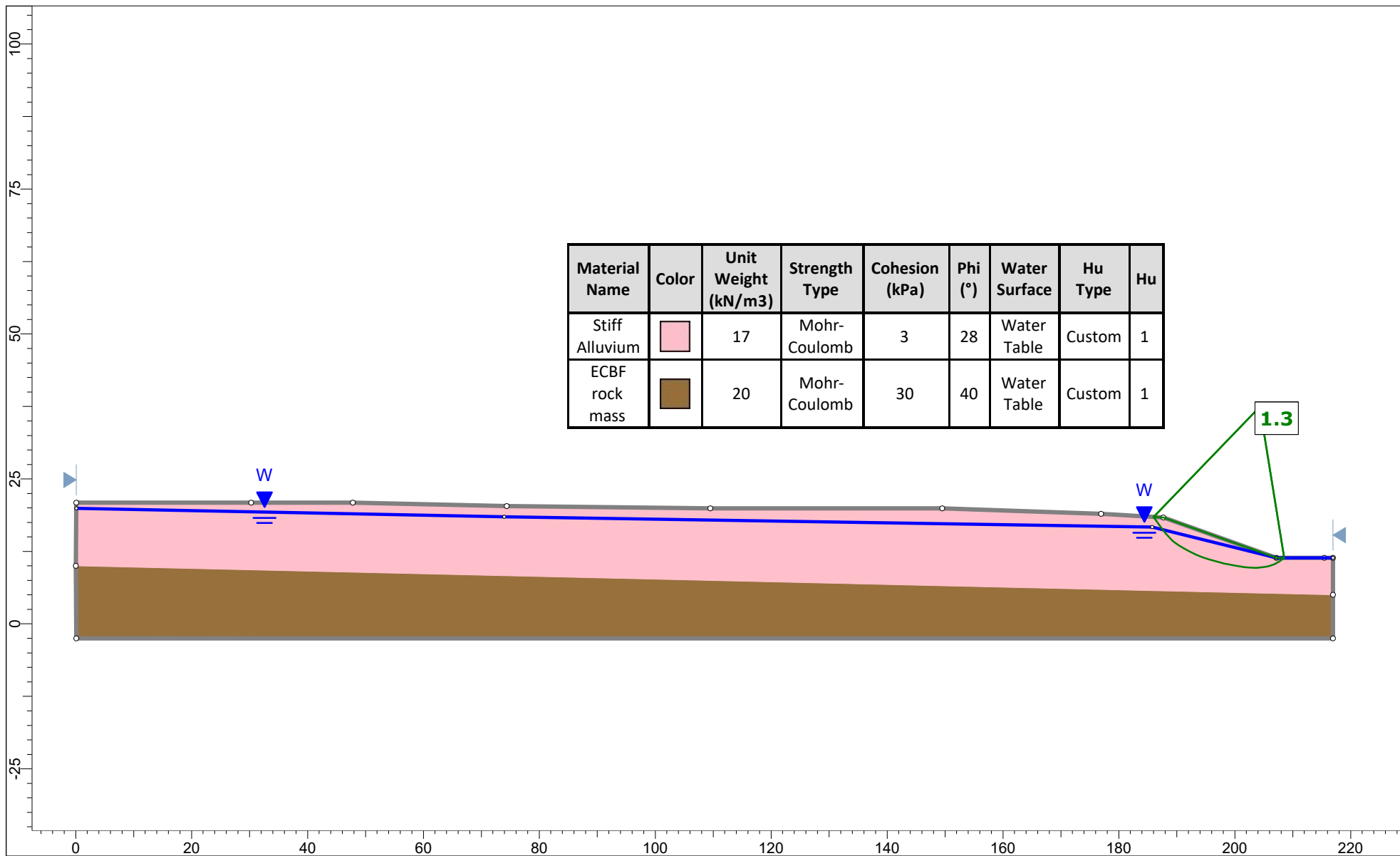


Material Name	Color	Unit Weight (kN/m ³)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type	Water Surface	Hu Type	Hu
Stiff Alluvium (UD)		20	Undrained	65	0	Constant	Water Table	Custom	0
Engineered Fill (UD)		18	Undrained	100	0	Constant	Water Table	Custom	0
ECBF Rock Mass (UD)		20	Undrained	1000	0	Constant	Water Table	Custom	0

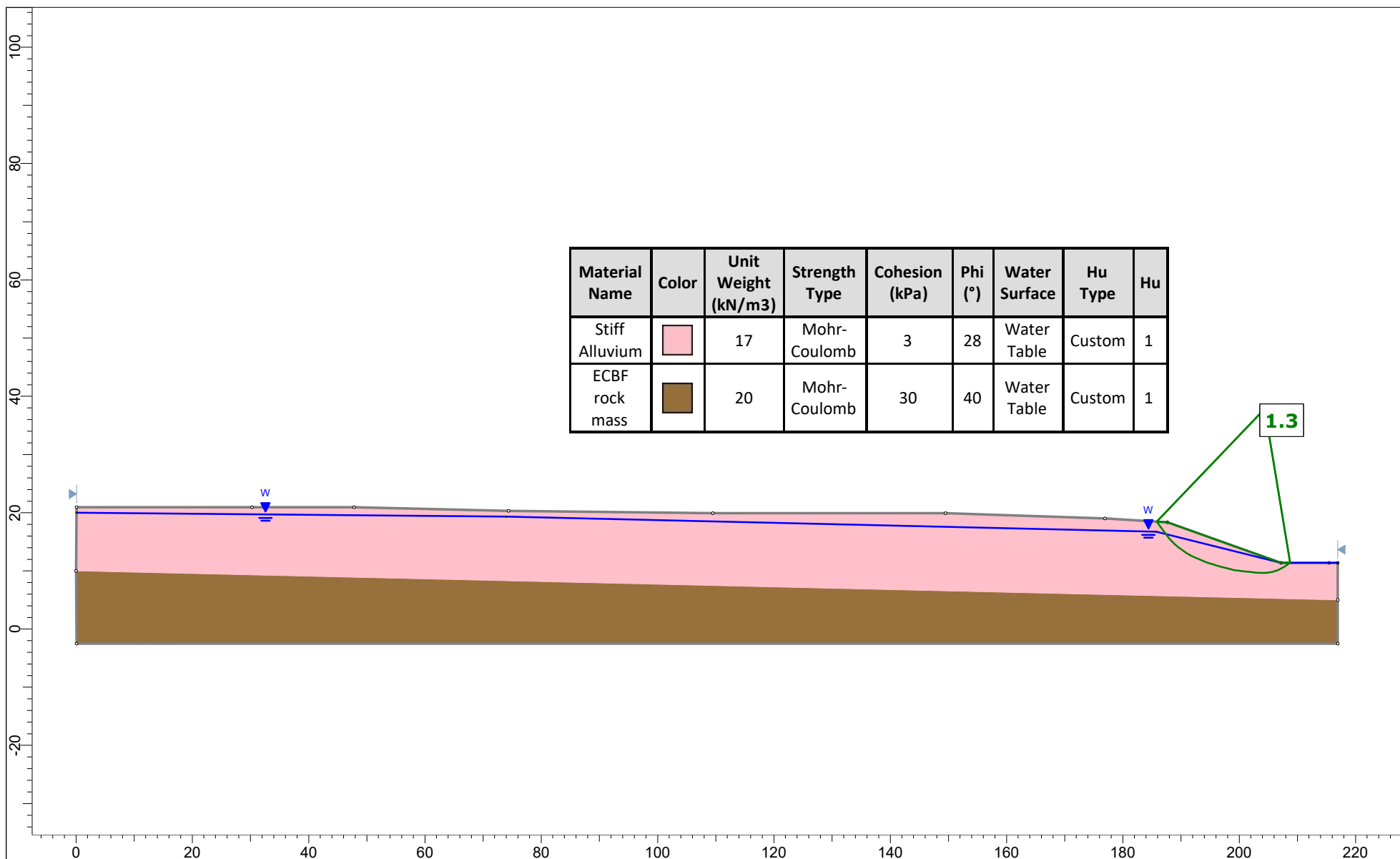


SLIDEINTERPRET 9.038

Project	Milldale Stages 14-17		
Group	Section XS N - Proposed	Scenario	Seismic
ST	CR	Company	Gibb Ritchie Geotechnical Engineers
Date	31/07/2026 3:36:06 pm	File Name	XS N.slm

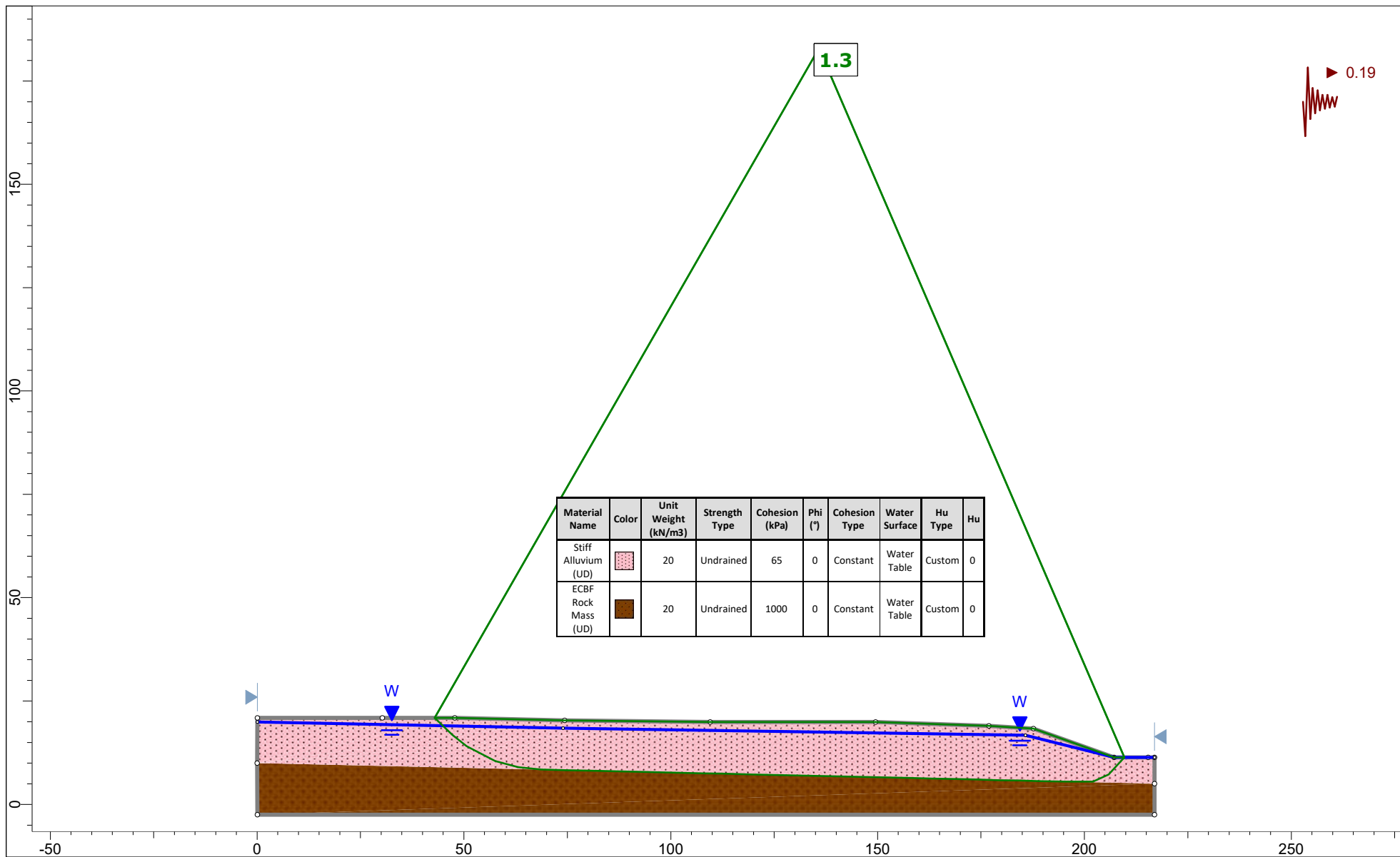


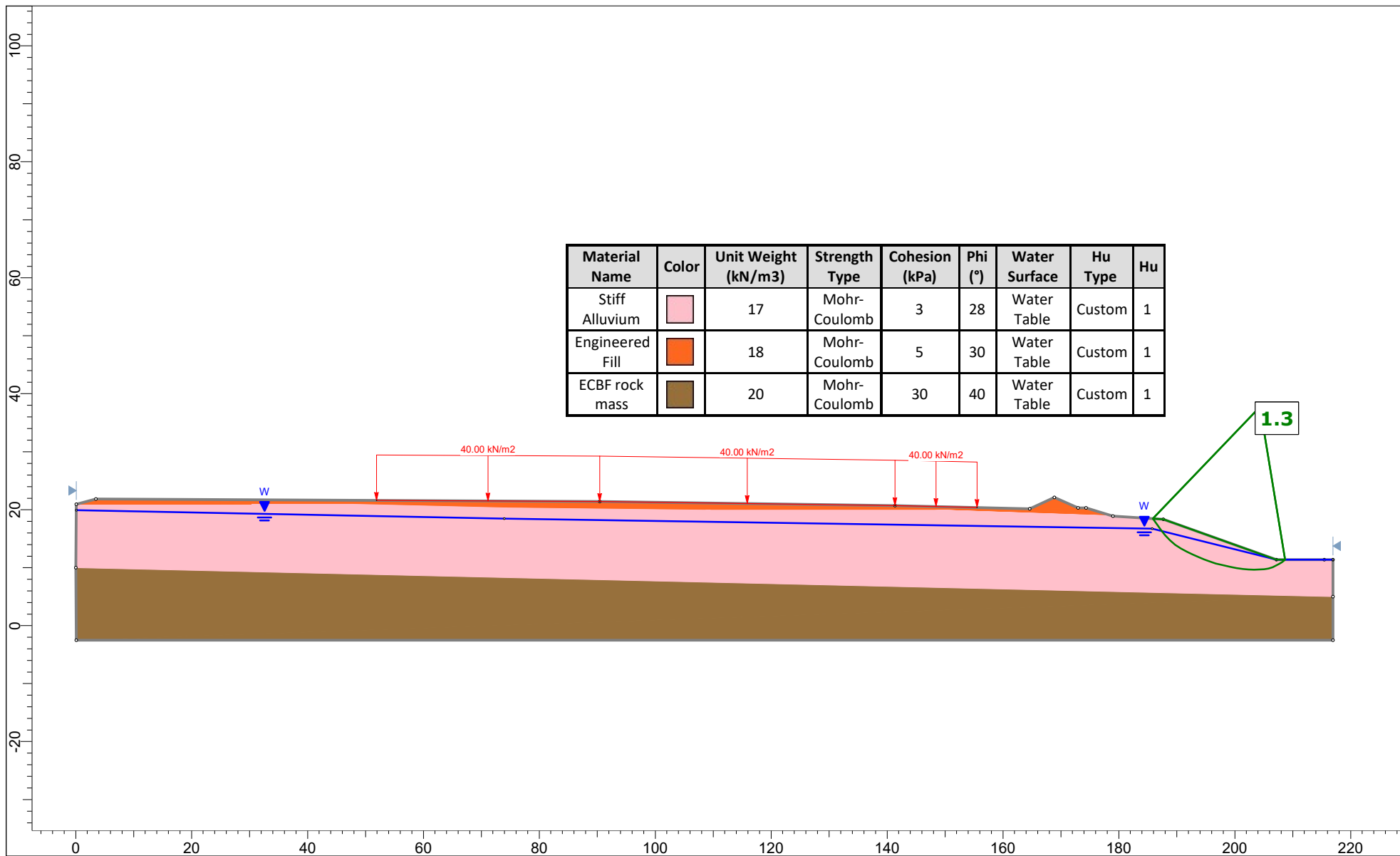
Project		XS O - Milldale Stages 14 - 17	
Group	Section XS O - Existing	Scenario	NGW
ST	CR	Company	Gibb Ritchie Geotechnical Engineers
Date	30/07/2026 11:16:57 am	File Name	XS O.slmd



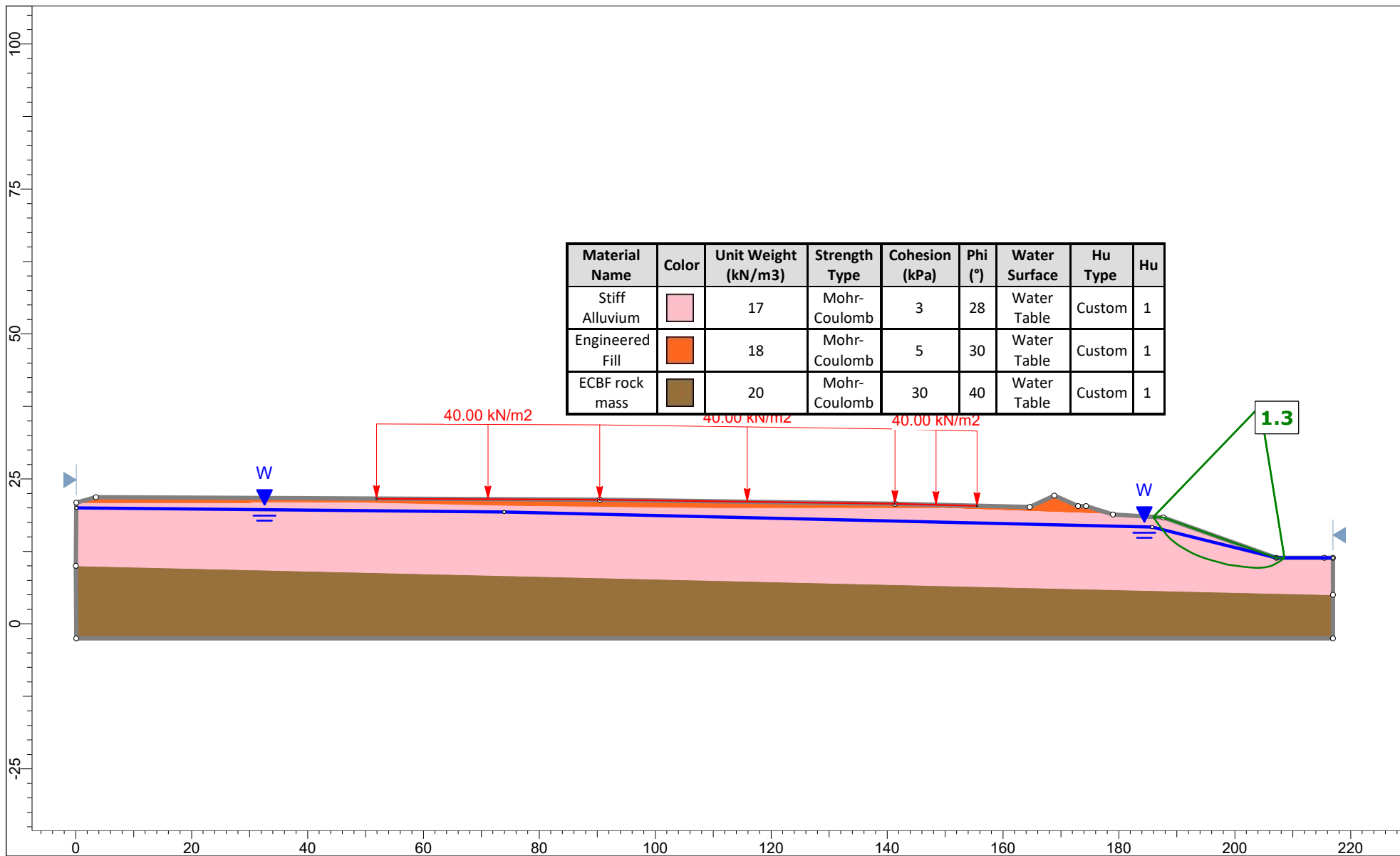
SLIDEINTERPRET 9.038

Project		XS O - Milldale Stages 14 - 17	
Group	Section XS O - Existing	Scenario	HGW
ST	CR	Company	Gibb Ritchie Geotechnical Engineers
Date	30/07/2026 11:16:57 am	File Name	XS O.slmd

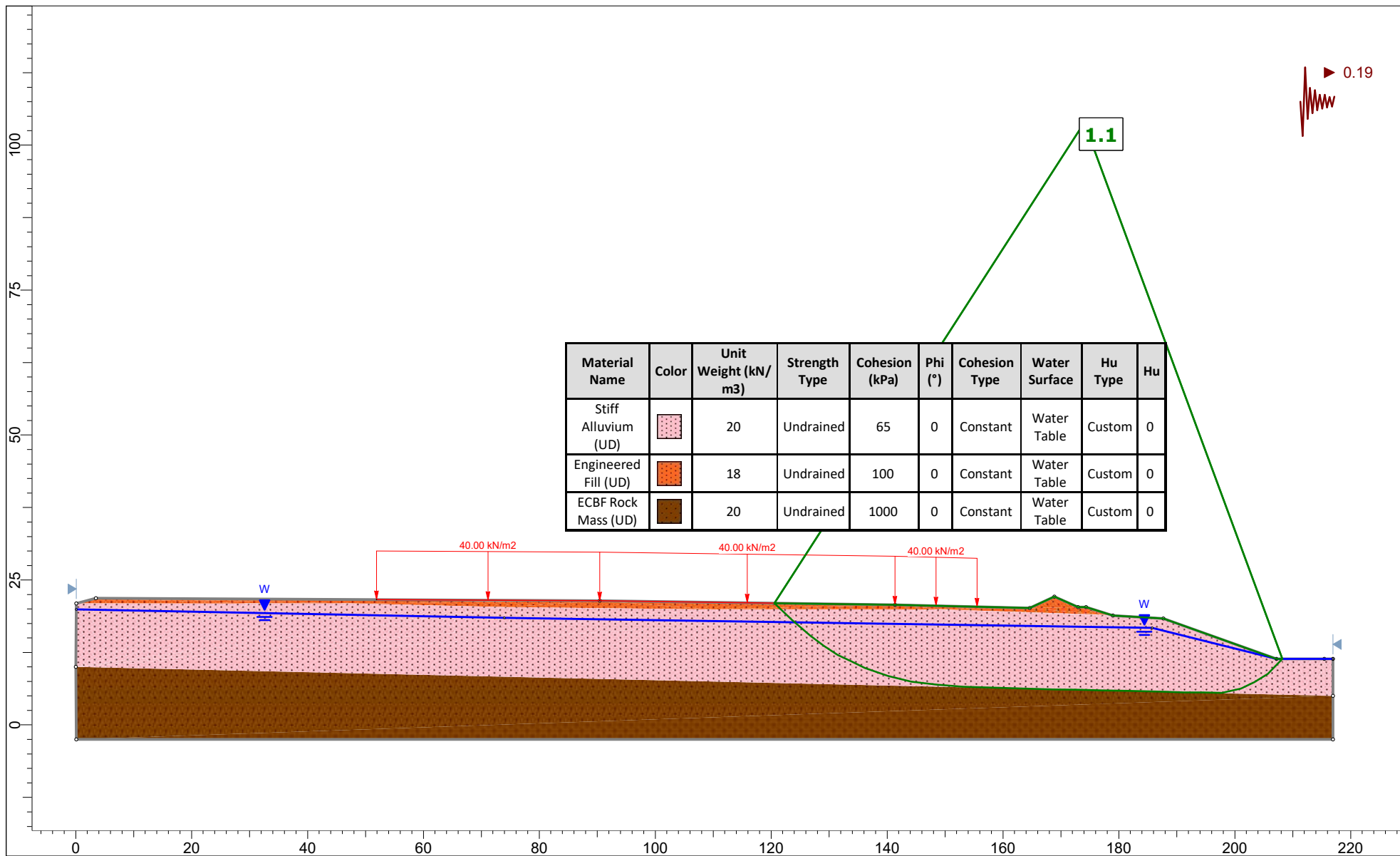




Project		XS O - Milldale Stages 14 - 17	
Group	Section XS O - Proposed	Scenario	NGW
ST	CR	Company	Gibb Ritchie Geotechnical Engineers
Date	30/07/2026 11:16:58 am	File Name	XS O.slmd



Project		XS O - Milldale Stages 14 - 17	
Group	Section XS O - Proposed	Scenario	HGW
ST	CR	Company	Gibb Ritchie Geotechnical Engineers
Date	30/07/2026 11:16:59 am	File Name	XS O.slm



SLIDEINTERPRET 9.038

Project

XS O - Milldale Stages 14 - 17

Group

Section XS O - Proposed

Scenario

Seismic

ST

CR

Company

Gibb Ritchie Geotechnical Engineers

Date

30/07/2026 11:16:59 am

File Name

XS O.slm

APPENDIX E

PC 120 Landslide Assessment



LANDSLIDE RISK ASSESSMENT

Milldale Water Infrastructure Facility

Fulton Hogan Land Development Ltd
AK26-027AC Rev.0

12 June 2026

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 Water Infrastructure Facility 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.

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2	LANDSLIDE RISK ASSESSMENT METHODOLOGY	3
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Revision History		
Date	Revision	Comments
12 June 2026	A	Internal Draft
15 June 2026	0	Draft for review

Signatures		
Name	Position	Signature
Prepared by	Chris Ritchie	Principal Geotechnical Engineer, CPEng
Reviewed by	Sam Gibb	Principal Geotechnical Engineer, CPEng

3 STAGE 1 DESKTOP STUDY AND HAZARD ASSESSMENT

3.1 Related Documents

A comprehensive desk-study assessment was undertaken reviewing the following sources:

- Auckland Council GeoMaps natural hazard layers
- Published landslide inventories
- Shallow and deep-seated landslide susceptibility maps
- Landslide runout susceptibility for areas within 150 m uphill of the site
- Aerial photography, LIDAR contours, and geomorphological interpretation

3.2 Landslide Susceptibility Maps

A comprehensive desk-study assessment was undertaken reviewing the following sources:

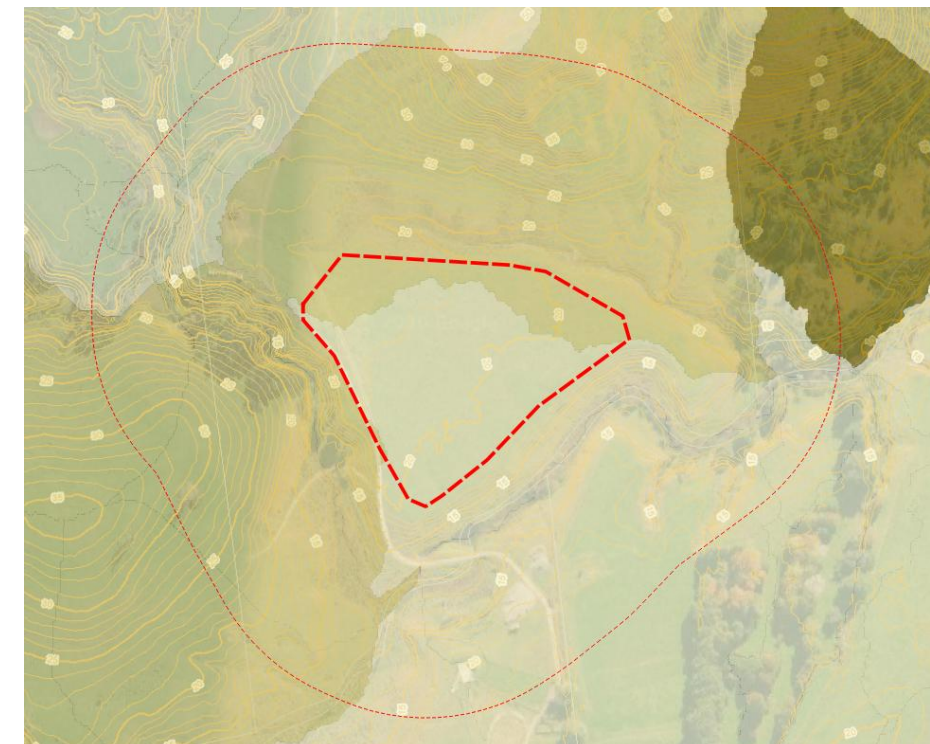
- Auckland Council GeoMaps natural hazard layers.
- Published landslide inventories.
- Shallow and deep-seated landslide susceptibility maps.
- Landslide runout susceptibility for areas within 150 m uphill of each stage.

Outputs of both shallow and deep-seated landslide susceptibility maps for each stage are shown in Figure 3. In summary:

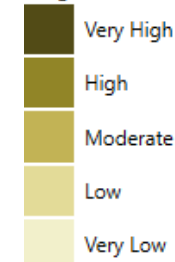
- Large scale landslide susceptibility is mapped as very low within the footprint of the facility,
- Shallow landslide susceptibility is mapped as generally very low.
- To determine run-out susceptibility, the highest landslide susceptibility class mapped within 150m of each stage was reviewed. This shows very high shallow landslide susceptibility on the neighbouring slopes above the facility and along the nearby water courses.

In summary the maps typically show very low to low shallow and large-scale landslide susceptibility. However, high to very high shallow and large-scale landslide susceptibility is present within 150m of the facility.

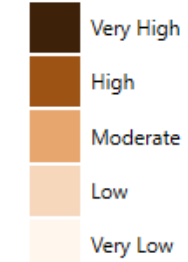
Figure3 Landslide Susceptibility Maps from Auckland Council GIS



Large Scale Landslide Susceptibility



Shallow Landslide Susceptibility



3.3 Mapped Existing Landslides

Two mapped landslides on Auckland Council GIS and Landslide Database. These have limited information, however they are present in steep topography above the site. The runout of these landslips is very short (less than 10m in length).

Figure 4 Mapped landslides (Auckland Council GIS)



4 STAGE 2 RISK ASSESSMENT METHOD SELECTION

Based on these findings, **Method 2** is required due to the proposed activity of “On-site septic tanks, wastewater treatment and disposal systems, effluent disposal fields, underground storage tanks, water tanks (including rainwater tanks) or stormwater pipes or soakage fields, accessways and private roads” and the mapped very shallow large scale landslide susceptibility.

4.1 Geological Model

The full geotechnical investigation and report has been completed for this project. This includes details on the engineering geological ground model, natural hazards assessments including numerical slope stability analysis and preliminary remediation design. The report reference is AK26-027AC Milldale Stages 14 -17 Water Infrastructure Facility Geotechnical Investigation Report, Gibb Ritchie Geotechnical Engineers Ltd, **dated 5 June 2026.**

5 STAGE 3 – METHOD 1

6 SEMI-QUANTITATIVE RISK ASSESSMENT

Three credible landslide hazard scenarios were assessed in accordance with Appendix 24:

Project:	Milldale Water Infrastructure Facility	Designed:	CR
Client:	Fulton Hogan Land Development Ltd	Checked:	SG
Project No:	AK26-027	Page:	1
Subject:	Appendix 24 - Landslide Risk Assessment	Date:	12/06/2026
Design Case:	Definition of Landslide Hazards		



Landslide Hazard Scenarios	
Scenario	Description
High	Shallow landslide occurring on steep slopes to the north, similar to the existing slips mapped.
Median	Shallow underslip to the south west on the gully flank.
Maximum Credible	Shallow underslip to the south into the Orewa River

Plan



Landslip Type	
A) Rock Fall	B) Rock Topple
C) Rotational Slide	D) Translational Slide
E) Lateral Spread	F) Debris Flow
G) Debris Avalanche	H) Soil Creep

Figure 4 Schematic cross-sections of different landslide types. (from Highland & Robinson, 2008)

Project:	Milldale Water Infrastructure Facility	Designed:	CR
Client:	Fulton Hogan Land Development Ltd	Checked:	SG
Project No:	AK26-027	Page:	1
Subject:	Appendix 24 - Landslide Risk Assessment	Date:	12/06/2026
Design Case:	Method - Qualitative Assessment		



Landslide Hazard Scenarios	
Scenario	Description
High	Shallow landslide occurring on steep slopes to the north, similar to the existing slips mapped.
Median	Shallow underslip to the south west on the gully flank.
Maximum Credible	Shallow underslip to the south into the Orewa River

Likelihood (Table 3, Appendix 24)			
Likelihood Category	Likelihood Descriptor	Annual Probability	Equivalent AEP
Almost certain	The event is expected to occur over the likely duration of the activity	10 ⁻¹	1 in 10 (10%)
Likely	The event will probably occur under adverse conditions over the likely duration of the activity	10 ⁻²	1 in 100 (1%)
Possible	The event could occur under adverse conditions over the likely duration of the activity	10 ⁻³	1 in 1,000 (0.1%)
Unlikely	The event might occur under very adverse circumstances over the likely duration of the activity	10 ⁻⁴	1 in 10,000 (0.01%)
Rare	The event is conceivable but only under exceptional circumstances over the likely duration of the activity	10 ⁻⁵	1 in 100,000 (0.001%)
Barely credible	The event is inconceivable or fanciful over the likely duration of the activity	10 ⁻⁶	1 in 1,000,000 (0.0001%)

Scenario	Justification	Value Adopted
High	Landslips of this nature have occurred, and therefore possible.	Possible
Median	Collapse of gully banks are unlikely to occur as these were not observed during Cyclone Gabrielle or other significant weather events. Slope stability modelling suggests this is unlikely.	Unlikely
Maximum Credible	Collapse of Orewa Banks are unlikely to occur as these were not observed during Cyclone Gabrielle or other significant weather events. Slope stability modelling suggests this is unlikely.	Unlikely

Consequence (Table 4, Appendix 24)				
Consequence Category	Human safety	Critical Buildings	Community buildings	Buildings Sensitive to Natural Hazards
Catastrophic	>10 dead and/or >1000 injured	Building unusable for >1 week	Building unusable for >1 week	Structure(s) completely destroyed and/or large-scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property major consequence damage.
Major	1-10 dead and/or 101-1000 injured	Evacuation of building required and/or building unusable for 1 week or less	Building unusable for 1 week to 1 month	Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.
Medium	0 dead, 11-100 injured	Building in landslide hazard risk assessment area but usability not affected	Evacuation of building required and/or building unusable for 1 week or less	Moderate damage to some of structure, and/or significant part of site requiring large stabilisation works. Could cause at least one adjacent property minor consequence damage.
Minor	0 dead, 1-10 injured	N/A	Building in landslide hazard risk assessment area but usability not affected	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.
Insignificant	0 dead, 0 injured	Building outside landslide hazard risk assessment area and usability not affected	Building outside landslide hazard risk assessment area and usability not affected	Little damage.

Scenario	Justification	Value Adopted
High	Platform is outside of the landslide hazard risk assessment area	Insignificant
Median	Platform is outside of the landslide hazard risk assessment area	Insignificant
Maximum Credible	Platform is outside of the landslide hazard risk assessment area	Insignificant

Risk Classification, Table 6 Appendix 24						
		Consequence Category				
		Insignificant	Minor	Medium	Major	Catastrophic
Likelihood Category	Almost certain	Medium (tolerable)	High (significant)	High (significant)	High (significant)	High (significant)
	Likely	Low (acceptable)	Medium (tolerable)	High (significant)	High (significant)	High (significant)
	Possible	Low (acceptable)	Low (acceptable)	Medium (tolerable)	High (significant)	High (significant)
	Unlikely	Low (acceptable)	Low (acceptable)	Low (acceptable)	Medium (tolerable)	High (significant)
	Rare	Low (acceptable)	Low (acceptable)	Low (acceptable)	Low (acceptable)	Medium (tolerable)
	Barely credible	Low (acceptable)	Low (acceptable)	Low (acceptable)	Low (acceptable)	Low (acceptable)

Scenario	Value Adopted
High	Low (acceptable)
Median	Low (acceptable)
Maximum Credible	Low (acceptable)

7 STAGE 3 – METHOD 2

8 QUANTITATIVE RISK ASSESSMENT

A quantitative assessment was undertaken using Appendix 24 Method 2 to calculate the Annual Individual Fatality Risk (AIFR) for three credible landslide hazard scenarios:

Scenario 1: Soil Creep on steeper slopes within the property (High)

Calculated AIFR: $4.17 \times 10^{-10} \rightarrow$ Low (acceptable)

Scenario 2: Shallow landslide on steeper slopes to the east within the gully flanks (Median)

Calculated AIFR: $1.39 \times 10^{-9} \rightarrow$ Low (acceptable)

Scenario 3: Maximum credible global slope failure of northern slope

Calculated AIFR: AIFR: $1.39 \times 10^{-9} \rightarrow$ Low (acceptable)

Combined AIFR

$$4.17 \times 10^{-10} + 1.39 \times 10^{-9} = 1.18 \times 10^{-9}$$

Note that Scenario 2 and 3 are from the same event so only one considered in sum.

All scenarios result in Low (acceptable) individual risk classifications under Appendix 24 Figure 5.

9 STAGE 4 – ACTIVITY STATUS AND PLANNING IMPLICATIONS

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

Under the Auckland Unitary Plan (Chapter E36), a low landslide hazard risk classification means that most proposed activities, including subdivision, can proceed as Permitted or Restricted Discretionary activities, provided that:

- A site-specific landslide hazard risk assessment prepared by a suitably qualified and experienced practitioner (SQEP) is submitted in accordance with E36.6 Standards and Appendix 24 methodology (this report).
- The development does not increase landslide risk beyond the site.
- Appropriate mitigation measures (engineered retaining walls, drainage, erosion and sediment controls, and surface water management) are implemented and maintained.

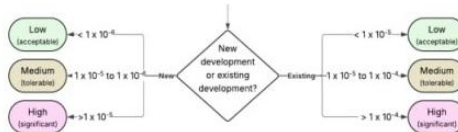
The proposed development therefore complies with the risk-based planning framework of E36 and meets the requirements for activities in natural hazard areas.

10 CLOSURE

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 Water Infrastructure Facility project. It provides geotechnical information and recommendations to assist in consenting.

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.

Project:	Milldale Water Infrastructure Facility	Designed:	CR	
Client:	Fulton Hogan Land Development Ltd	Checked:	SG	
Project No:	AK26-027	Page:	1	
Subject:	Appendix 24 - Landslide Risk Assessment	Date:	12/06/2026	
Design Case:	Method 2 - Quantitative Assessment			
Landslide Hazard Scenarios				
Scenario	Description			
High	Shallow landslide occurring on steep slopes to the north, similar to the existing slips mapped.			
Median	Shallow underslip to the south west on the gully flank.			
Maximum Credible	Shallow underslip to the south into the Orewa River			
Quantitative Risk Assessment				
Calculate Annual Individual Fatality Risk (AIFR) refer to calculation				
$P_{(LoL)} = P_{(H)} \times P_{(S:H)} \times P_{(T:S)} \times V_{(D:T)}$ Where $P_{(LoL)}$ is the annual probability of loss of life (death) of an individual. $P_{(H)}$ is the annual probability of the landslide occurring. $P_{(S:H)}$ is the probability of spatial impact by the landslide on the property, taking into account the travel distance and travel direction of the given event. $P_{(T:S)}$ is the temporal spatial probability (e.g. of the building or location being occupied by the individual) given the spatial impact and allowing for the possibility of evacuation given there is warning of the landslide occurrence. $V_{(D:T)}$ is the vulnerability of the individual (probability of loss of life of the individual given the impact). 				
Scenario	High			
	Value	Justification		
$P_{(H)}$	0.01	Likely to occur during 1 in 100 year storm event		
$P_{(S:H)}$	0.001	Extremely unlikely for runoff to reach the facility		
$P_{(T:S)}$	0.041666667	Typically unmanned plant, assume visitor for 1 hour per day		
$V_{(D:T)}$	0.001	Unlikely to reach plant buildings were the occupant would be		
$R_{(LoL)}$	4.17E-10			
Susceptibility	Low (acceptable)			
Scenario	Median			
	Value	Justification		
$P_{(H)}$	0.003333333	Assume that landslide could occur during 1 in 300 year storm (did not occur during anniversary weekend event)		
$P_{(S:H)}$	0.01	Plant buildings are well offset from this bank		
$P_{(T:S)}$	0.041666667	Typically unmanned plant, assume visitor for 1 hour per day		
$V_{(D:T)}$	0.001	Underslip likely to only affect leading edge of a building, all well set back from the slope		
$R_{(LoL)}$	1.39E-09			
Susceptibility	Low (acceptable)			
Scenario	Maximum Credible			
	Value	Justification		
$P_{(H)}$	0.003333333	Assume that landslide could occur during 1 in 300 year storm (did not occur during anniversary weekend event)		
$P_{(S:H)}$	0.01	Plant buildings are well offset from this bank		
$P_{(T:S)}$	0.041666667	Typically unmanned plant, assume visitor for 1 hour per day		
$V_{(D:T)}$	0.001	Underslip likely to only affect leading edge of a building, all well set back from the slope		
$R_{(LoL)}$	1.39E-09			
Susceptibility	Low (acceptable)			

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 Water Infrastructure Facility 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.

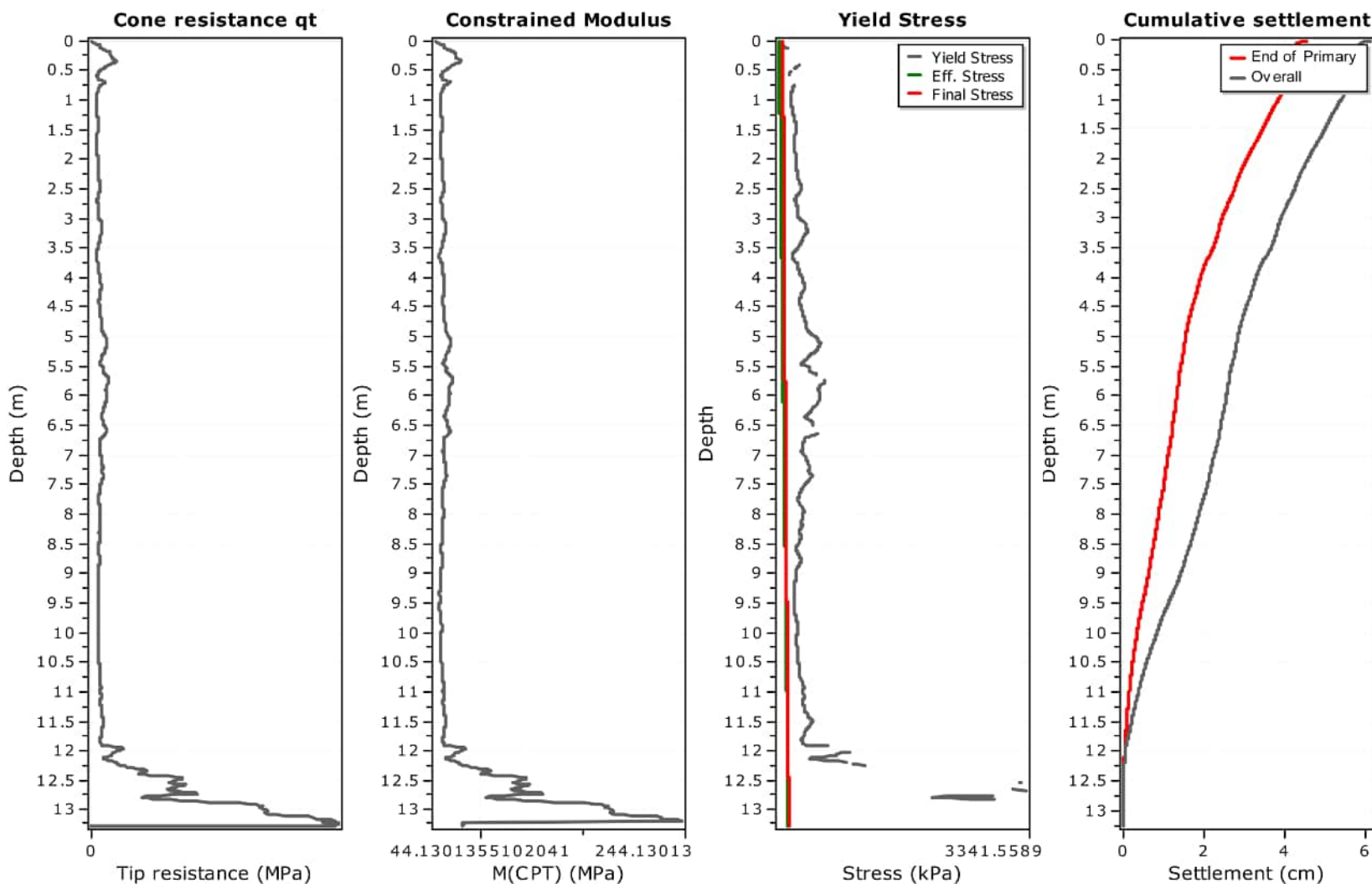
If you have any questions about the content or use of this report, please contact Gibb Ritchie Geotechnical Engineers for clarification.



APPENDIX F

CPT-IT Static Settlement Outputs

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Circular
Footing diameter: 10.00 (m)
L/B: 1.0
Footing pressure: 45.00 (kPa)
Embedment depth: 0.00 (m)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 12 months
Time period for second. settlements: 588 months

* Primary settlement calculation is performed according to the following formula:

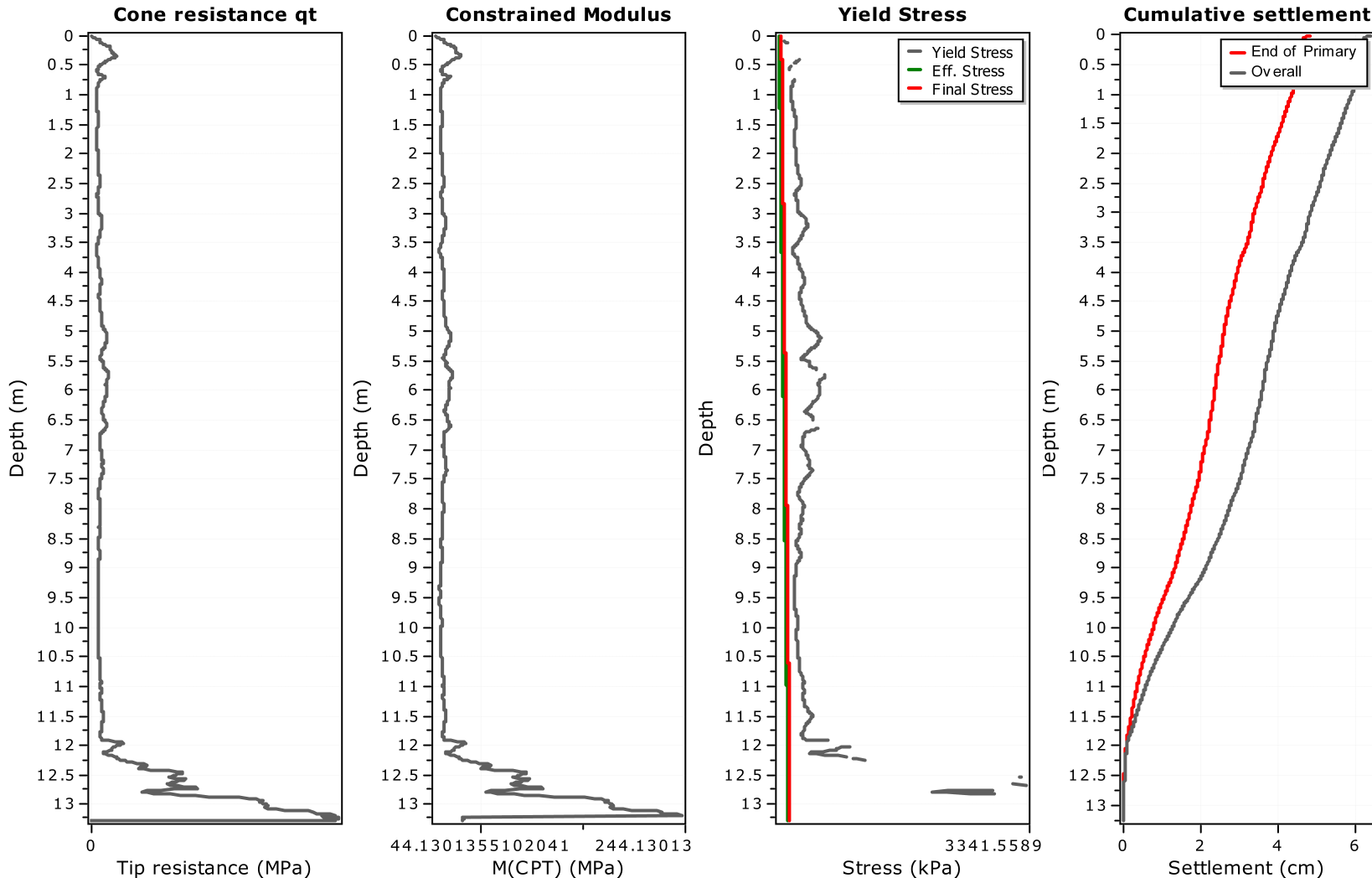
$$S = \sum \frac{\Delta \sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S_{sec} = S_{pri} \left(1 - e^{-\frac{t}{t_p}} \right)$$

where t_p is the duration of primary consolidation

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Circular
Footing diameter: 35.00 (m)
L/B: 1.0
Footing pressure: 30.00 (kPa)
Embedment depth: 0.00 (m)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 12 months
Time period for second. settlements: 588 months

* Primary settlement calculation is performed according to the following formula:

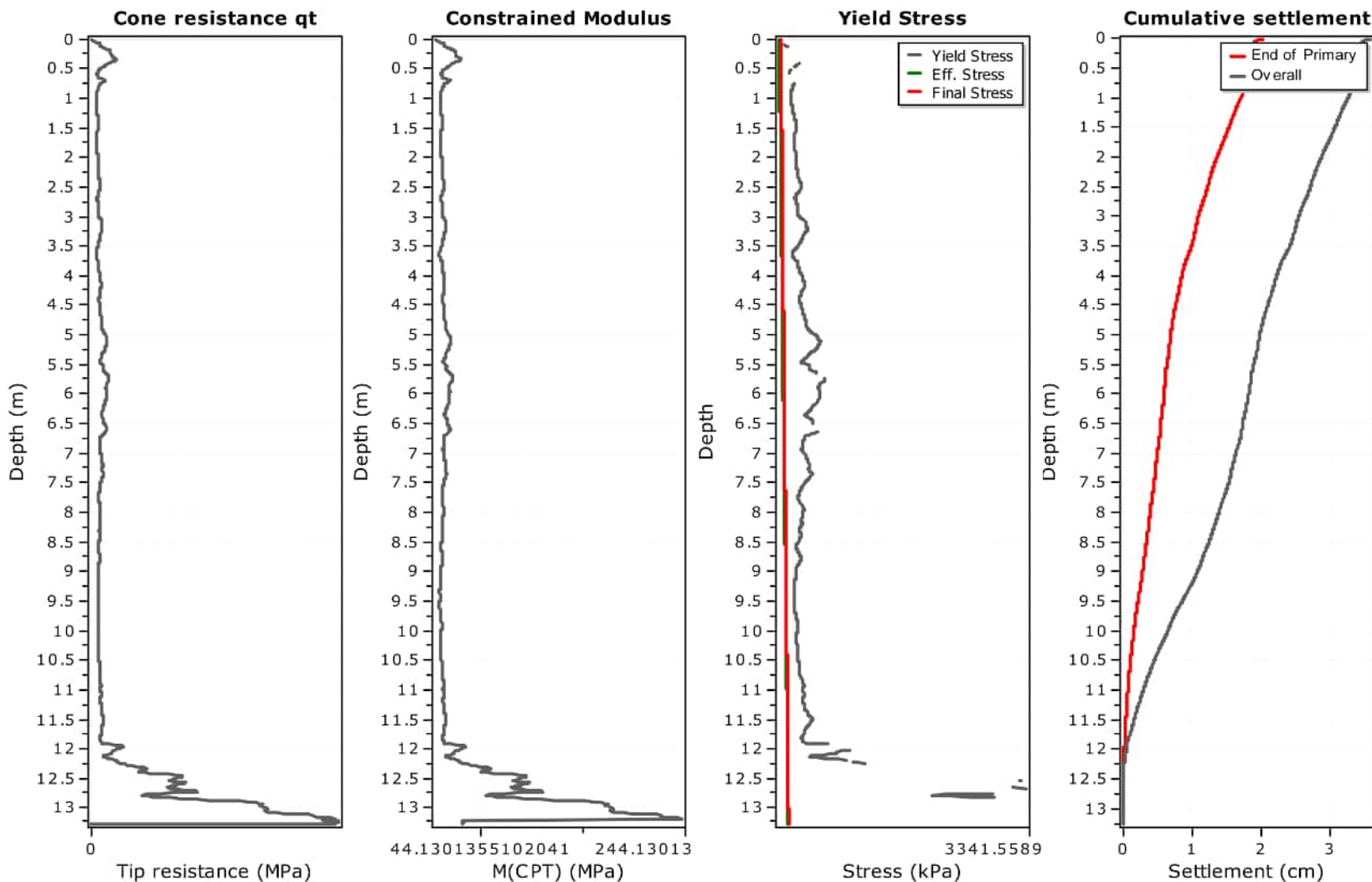
$$S = \sum \frac{\Delta \sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S_{sec} = S_{pri} \left(1 - e^{-\frac{t}{t_p}} \right)$$

where t_p is the duration of primary consolidation

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Circular
Footing diameter: 10.00 (m)
L/B: 1.0
Footing pressure: 20.00 (kPa)
Embedment depth: 0.00 (m)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 12 months
Time period for second. settlements: 588 months

* Primary settlement calculation is performed according to the following formula:

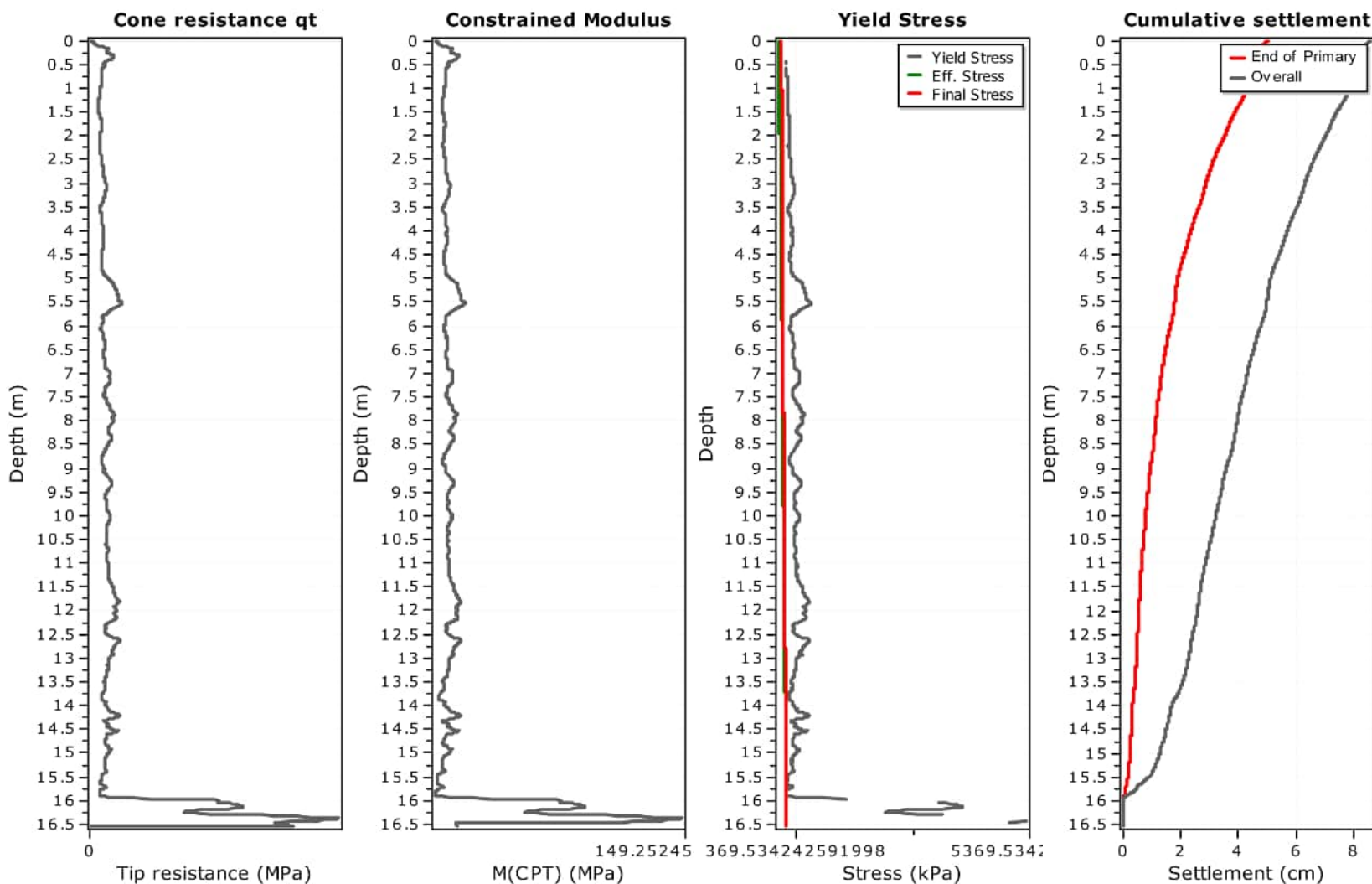
$$S = \sum \frac{\Delta \sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S_{sec} = S_{pri} \left(1 - e^{-\frac{t}{t_p}} \right)$$

where t_p is the duration of primary consolidation

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Circular
Footing diameter: 10.00 (m)
L/B: 1.0
Footing pressure: 45.00 (kPa)
Embedment depth: 0.00 (m)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 6 months
Time period for second. settlements: 588 months

* Primary settlement calculation is performed according to the following formula:

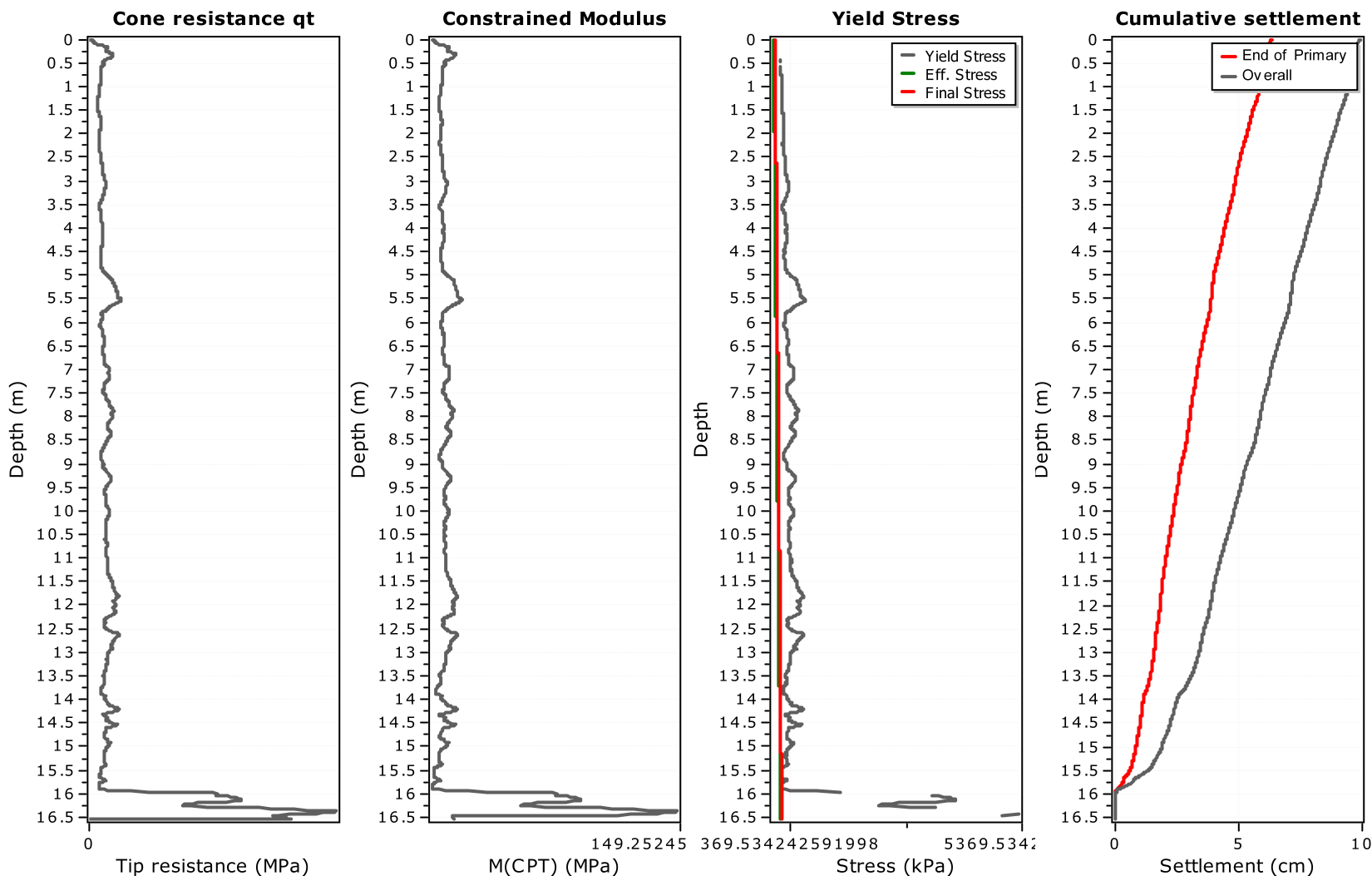
$$S = \sum \frac{\Delta \sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S_{sec} = S_{pri} \left(1 - e^{-\frac{t}{t_p}} \right)$$

where t_p is the duration of primary consolidation

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 35.00 (m)
L/B: 1.0
Footing pressure: 30.00 (kPa)
Embedment depth: 0.00 (m)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 6 months
Time period for second. settlements: 588 months

* Primary settlement calculation is performed according to the following formula:

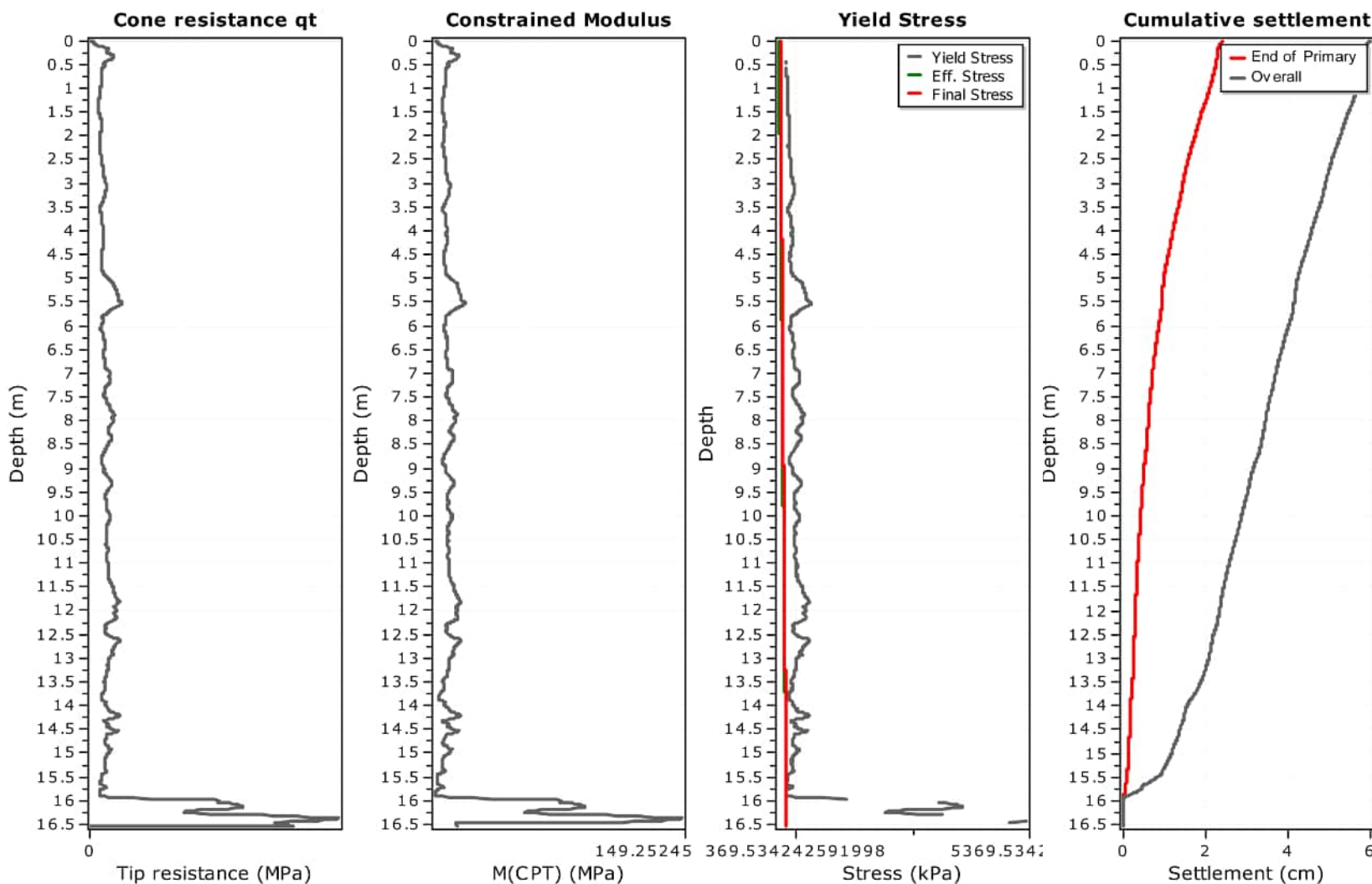
$$S = \sum \frac{\Delta \sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S_{sec} = S_{pri} \left(1 - e^{-\frac{t}{t_p}} \right)$$

where t_p is the duration of primary consolidation

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 10.00 (m)
L/B: 1.0
Footing pressure: 20.00 (kPa)
Embedment depth: 0.00 (m)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 6 months
Time period for second. settlements: 588 months

* Primary settlement calculation is performed according to the following formula:

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

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S_{sec} = S_{pri} \left(1 - e^{-\frac{t}{t_p}} \right)$$

where t_p is the duration of primary consolidation

APPENDIX G

Risk Assessment

	CLIENT:	FULTON HOGAN LAND DEVELOPMENT LTD	DESIGNER:	CR
	PROJECT:	MILLDALE STAGES 14-17 - WATER INFRASTRUCTURE	CHECKED:	SG
	TITLE:	GEOTECHNICAL RISK REGISTER	JOB NO:	AK26-027
			DATE:	17/06/2026
			ISSUED FOR:	GIR

Item	Geotechnical Hazard	Description	Relevant Standards*/Guidance Documents	Area Assessed	Assessment Outcome	Existing Risk of Damage to Land / Structures			Mitigation Measure	Residual Risk of Damage to Land / Structures		
						Likelihood	Consequence	Risk Rating		Likelihood	Consequence	Risk Rating
1	Earthquake	Seismicity	NZGS and MBIE (2021) Earthquake Geotechnical Engineering Practice in New Zealand, Module 1: Overview of the Guidelines Structural design actions - Part 0: General principles, NZS1170.0:2002 Structural design actions - Part 5: Earthquake actions - New Zealand, NZS1170.5:2004	Entire Site	Site subsoil class = Class C due to depth of soils SLS PGA = 0.07g, ULS PGA = 0.19g			0				
		Fault Rupture	GNS Science, New Zealand Active Faults Database, retrieved from https://data.gns.cri.nz/af/	Entire Site	Nearest active fault = Wairoa North Fault, approximately 60km from the site. Recurrence interval of 13,000 years.	2	4	8	Not required	2	4	8
		Liquefaction	MfE and MBIE (2017), Planning and engineering guidance for potentially liquefaction-prone land NZGS and MBIE (2021) Earthquake Geotechnical Engineering Practice in NZ, Module 1: Overview of the Guidelines NZGS and MBIE (2021) Earthquake Geotechnical Engineering Practice in NZ, Module 2: Geotechnical Investigations for Earthquake Engineering NZGS and MBIE (2021) Earthquake Geotechnical Engineering Practice in NZ, Module 3: Identification, assessment and mitigation of liquefaction hazards MBIE (2012) Repairing and rebuilding houses affected by the Canterbury earthquakes, Part A: Technical Guidance Robertson, P. K. (2015) Comparing CPT and Vs Liquefaction Triggering Methods, J. Geotech. Geoenviron. Eng. Bray, J. D. & Macedo, J. (2017) Simplified procedure for estimating liquefaction-induced building settlement Auckland Council Geomaps	Entire Site	Maps show the site is within a very low to unlikely hazard zone for liquefaction vulnerability. Intrusive investigations including lavatory testing suggests the underlying soils are generally cohesive and therefore at low risk of liquefaction. Specific analysis using CLIQ software suggests low liquefaction risk and associated settlements are expected to be low (maximum 25mm ULS). Refer to Section 7.2 of the report.	2	4	8	Mitigation not required.	2	4	8
		Lateral Spread	NZGS and MBIE (2021) Earthquake Geotechnical Engineering Practice in NZ, Module 3: Identification, assessment and mitigation of liquefaction hazards MBIE (2012) Repairing and rebuilding houses affected by the Canterbury earthquakes, Part A: Technical Guidance	Entire Site	No lateral spread anticipated as no liquefaction expected.	1	4	4	Mitigation not required.	1	4	4
		Cyclic Softening	NZGS and MBIE (2021) Earthquake Geotechnical Engineering Practice in NZ, Module 3: Identification, assessment and mitigation of liquefaction hazards	Entire Site	Cyclic shear strength of 80% of peak shear strength for stiff alluvium.	2	3	6	Mitigation not required.	2	3	6
		Global Instability	Auckland Council (2023), The Auckland Code of Practice for Land Development and Subdivision, Chapter 2: Earthworks and Geotechnical, Version 2	Entire site	Target Min. FOS = 1.5, 1.3 and 1.0 for prevailing, worst credible and seismic conditions. Results show that proposed landforms achieve target factors of safety. Refer Section 7.4 for calculation package and detailed results.	4	4	16	Mitigation not required	1	4	4
	Slope Instability /	Soil Creep		Entire site	Soil creep on slopes steeper than 1V:4H within upper 1m of ground surface, however no slopes within the platform.	4	1	4	Mitigation not required	4	1	4

	CLIENT:	FULTON HOGAN LAND DEVELOPMENT LTD	DESIGNER:	CR
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	TITLE:	GEOTECHNICAL RISK REGISTER	JOB NO:	AK26-027
			DATE:	17/06/2026
			ISSUED FOR:	GIR

Item	Geotechnical Hazard	Description	Relevant Standards*/Guidance Documents	Area Assessed	Assessment Outcome	Existing Risk of Damage to Land / Structures			Mitigation Measure	Residual Risk of Damage to Land / Structures		
						Likelihood	Consequence	Risk Rating		Likelihood	Consequence	Risk Rating
2	Slope instability / Landslide	Cut / Fill Batter Stability		Entire site	No batter slopes proposed	1	4	4	Mitigation not required	1	4	4
		Stream bank instability and erosion		Slopes adjacent to Orewa River	Refer to global stability analysis in respect to development near Orewa River and gullies. Stability models meet factor of safety requirements.	1	4	4	Mitigation not required	1	4	4
3	Problematic Soils	Pumice Soil Exposure	N/A	Not in this geology								
		Rockmass Exposure	N/A		Not anticipated as part of the works	1	3	3	Mitigation not required	1	3	3
		Expansive Soils	MBIE (2021) Acceptable Solutions and Verification Methods For New Zealand Building Code Clause B1 Structure, Amendment 20 NZS 3604:2011 AS 2870-2011 NZS 4402:1998 Test 2.1, 2.4 and 2.6 Fraser Thomas Ltd, Brown, B.J., Goldsmith, P.R., Shorten, J.P.M. & Henderson, L (2003) Soil Expansivity in the Auckland Region. BRANZ, Study Report SR 120	Entire Site	Experience on previous stages of Milldale suggest that expansive soil class will be between M and H1	4	4	16	Specific foundation design to be undertaken by structural engineer in accordance with AS 2870-2011. Final expansive class to be determined on a lot by lot basis during GCR stage.	4	1	4
		Sensitive Soils	NZGS Field Description of Soil and Rock (NZGS, 2005)	Entire Site	Hand shear vane testing suggests that natural soils are unlikely to be sensitive.	3	1	3	Mitigation not required.	1	3	3
		Acid Sulphate Soils		Not in this geology								
		Collapsible / Dispersible Soils	N/A	Not in this geology								
		Uncontrolled Fill	N/A	Entire Site	Not anticipated based on aerial photography	1	4	4	Uncontrolled fill is to be excavated and replaced during construction.	1	4	4
		Contamination	Ministry for the Environment (2021) Contaminated land management guidelines	Entire Site	By others							
4	Settlement	Compressible Soils	Mesri G, Kwan Lo D, Feng T (1994) Settlement of Embankments on Soft Clays, Chapter of Vertical and Horizontal Deformation of foundations and Embankments, A.T.Yeung & G.Y.Felio, ASCE, New York, 8-56 Mesri G, Ajlouni N (2007) Engineering Properties of Fibrous Peats, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 133:7. ASCE	Firm and soft alluvium	Total settlements of 60 to 150mm based on building typology predicted. Differential settlements expected to be beyond building code limits in this geology without specific mitigation. Refer to Section 7.5 in GIR for details	5	3	15	Several proposed mitigations methods including preload surcharge, ground improvement or piled structures are recommended.	2	3	6
		Bearing Capacity	MBIE (2021) Acceptable Solutions and Verification Methods For New Zealand Building Code Clause B1 Structure, Amendment 20, B1/VM4	Entire Site	Bearing pressures and foundation widths are unknown at this stage, however are likely to exceed the bearing capacity of the soils in large tank structures	5	3	15	Refer Section 8.5 of the report, specific design of foundations required.	2	3	6
		Effects of Dewatering			By Others.							

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			DATE:	17/06/2026
			ISSUED FOR:	GIR

Item	Geotechnical Hazard	Description	Relevant Standards*/Guidance Documents	Area Assessed	Assessment Outcome	Existing Risk of Damage to Land / Structures			Mitigation Measure	Residual Risk of Damage to Land / Structures		
						Likelihood	Consequence	Risk Rating		Likelihood	Consequence	Risk Rating
		Settlement Behind Retaining Wall – Boundary Effects		Entire Site	No boundary walls in scheme							
		Tomos			Not observed in this geology							
7	Geothermal activity	Formation of geysers, hot springs, fumaroles, mud pools	Auckland Council GeoMaps (Geothermal overlay)	Entire Site	Not in this geology							
9	Construction Risks	Excavatability	Transit New Zealand (1997) Specification for Earthworks Construction TNZ F/1:1997	Entire Site	Given the stiffness and sheared fabric of the soil/rock units that will be encountered, excavation is expected to be readily achieved with normal earthworks plant.							
		Sediment Retention Ponds	N/A	Sloping areas of site	SRP locations unknown, any SRPs on slopes may cause slope instability due to base seepage or placement of earthbund.	3	4	12	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.	1	4	4
		Stockpile locations	N/A	Orewa River and Waterloo Stream	Stockpile locations are currently unknown, any near the Orewa River or Waterloo Creek on soft to firm alluvium ground must be reviewed by the Geotechnical Engineer due to the potential of slope instability caused by rapid filling.	4	3	12	The location of all temporary stockpiles must be approved by the Geotechnical Engineer prior to placement. Where stockpiles cannot be avoided above sloping ground they should be placed over a wide area with the height restricted under the direction of the Geotechnical Engineer.	2	3	6
		Subgrade Preparation	N/A	Firm to soft alluvium	Soft to firm alluvium may be difficult to work on due to punching of subgrade and wet conditions	4	2	8	Consider leaving topsoil in place in reserve areas, undercut and replacement to create bridging layer during earthworks	2	2	4
		Service Trenches (trench collapse / long term settlement)	N/A	Firm to soft alluvium	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.	4	4	16	The contractor must consider these soils as "complex" and consider trench support or battering in this area.	2	4	8