REPORT

Geotechnical Assessment Report

Pakowhai Secondary Stopbank

Prepared for
Hawkes Bay Regional Council

Prepared by
Tonkin & Taylor Ltd

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Executive summary

This Geotechnical Assessment Report (GAR) provides interpretation to our investigations and commentary on the geotechnical aspects of the preliminary design. This includes analyses for the static, flooding (including transient and steady-state seepage), and SLS/ULS seismic performance of the proposed Pakowhai Secondary Stopbank. The overall performance of the earthen stopbanks is presented via high-level zoning due to the significant alignment length and the corresponding granularity of investigation testing.

For interpretation and assessment purposes the alignment is grouped into five geotechnical zones reflecting similar shallow foundation conditions and groundwater depths along the length of the stopbank. Shallow foundation soils in the northern areas generally coarser (sands/gravels) influencing landward seepage; southern areas tend to have finer near-surface soils (lower landward seepage potential), but higher liquefaction susceptibility. The stream diversion (CH3650 to 4000) area is governed by very shallow groundwater.

Static stability generally meets HBRC criteria across the alignment. Under design flood loading (200 m^3/s inflow from the Tutaekuri river with a 0.5% AEP river flood level in the Ngaruroro Overflow channels), transient performance is acceptable across assessed sections. However, two areas (GZ-01 – Northern Alignment and (GZ-05 - stream diversion) do not currently meet steady-state seepage criteria, and both approach critical hydraulic gradients. Under seismic loading, SLS damage effects are expected to be minor to moderate, while ULS damage effects could be moderate to severe in parts of the alignment. Selected stability cases under liquefied conditions do not currently meet HBRC targets. This covers GZ-03 (Stopbank 3B50, CH 1820 to 2380, CH 3620 to 3740, and CH 3940 to 4000) and GZ-05 (CH4520 to CH5929).

Where corridor width is limited by existing infrastructure (e.g. kiwifruit canopy structures and sheds), localised retaining/ground-improvement solutions will be required. These have had a preliminary assessment undertaken with site-specific ground models, noting the geotechnical zoning for the earthen stopbanks do not apply. The preliminary options include:

- (i) a hybrid low-height stopbank with a sheetpile flood wall upstand (to cover the difference to design level) – This is constrained to the FEL Block area (Stopbank 3B4C, CH900 to CH1700).
- (ii) river-side ground improvement with a landward retaining element or a river-side retaining structure with a deadman anchor (Stopbank 3B50, CH2060 to CH2100).

The applicable importance level for any wall should be agreed with HBRC at detailed design noting heavy vehicle surcharge loading (from earthworks vehicles) may govern design.

Based on the results of our ground investigations and seepage modelling, we recommend the following:

- Carrying out a detailed design with the assumption of landward toe drains (assumed 600 mm thick, 6 m long sand envelope) in zones nearing critical exit gradients (GZ-01 and GZ-05) to mitigate piping risk.
- Use a higher proportion of lower-permeability fill (silty sands) over blended river gravels to reduce overall stopbank permeabilities. A stopbank permeability of 1 x 10^{-6} m/s has been assumed for our design.
- Design development: Maintain the zone-based design approach and tailor measures to local separable portions.
- HBRC to confirm importance level for proposed floodwalls, and engage HBRC's engineering team to confirm geometry in constrained areas.

We understand HBRC will confirm borrow material sources at detailed design. For separable portion 1 (SP1), we understand HBRC will utilise fill material sourced from the True Left of the Ngaruroro River near the downstream extent of the stopbank. We understand this has been investigated by others for material characterisation. For other areas along the alignment, we understand much of this fill may be sourced from the external Matuku Rd. or AKM quarry sites.

1 Introduction

This report presents interpretation of ground investigations and a preliminary geotechnical assessment completed by Tonkin & Taylor Ltd (T+T) for the construction of the Pakowhai Secondary Stopbank. The work described in this document was commissioned by Hawke's Bay Regional Council (HBRC) and has been completed in accordance with the terms and conditions outlined in T+T's Variation Order 09: Pakowhai Flood Protection Improvements – Preliminary Design, dated 2 April 2025[1].

This report builds upon previous work associated with the Pakowhai Secondary Stopbank development including our:

- Initial concept design drawings, dated 10 December 2024[2].
- Initial preliminary schedule of quantities, dated 28 February 2025[3].
- Geotechnical factual reporting summarising our ground investigations to date, dated 21 March 2025[4].
- Contamination Preliminary site investigation and Detailed Site Investigation reporting, dated 9 September 2024 and 12 March 2025, respectively[5][6].
- Our optioneering report detailing flood protection options for pricing assessment, dated 30 May 2025[7].
- The Pakowhai flooding effects reporting showing the sensitivity of various flood models assessed., dated 22 August 2025[8].

This report should be read in conjunction with our preliminary design report, dated 11 September 2025[9].

1.1 Scope of work

T+T has carried out the following scope of work for the purposes of this investigation and assessment of the proposed stopbank:

- Undertaking a background geotechnical review of information provided by HBRC, and readily available geotechnical information both held by T+T and publicly accessible.
- Review of geotechnical investigations completed in 2024, the results of which are presented in our Geotechnical Factual Report[4].
- Preparation geological long sections (and representative cross sections) and a summary of the interpreted geological model based on each zone.
- An assessment of:
  - Seismic hazards along the alignment.
  - Static settlement of the proposed stopbanks.
  - The proposed stopbank stability under static, seismic and rapid drawdown conditions.

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[1] T+T (2025) *Variation Order 09: Pakowhai Flood Protection Improvements – Preliminary Design,* dated 2 April 2025.
[2] T+T (2025) *Pakowhai Secondary Stopbank – concept design,* dated 10 December 2024.
[3] T+T (2025) *Stopbanks enabling works draft schedule for pricing,* dated 28 February 2025.
[4] T+T (2025) *Pakowhai Secondary Stopbank – Geotechnical Factual Report,* dated 21 March 2025.
[5] T+T (2025) *Pakowhai Secondary Stopbank – Preliminary Site Investigation,* dated 9 September 2024.
[6] T+T (2025) *Pakowhai Secondary Stopbank – Detailed Site Investigation,* dated 13 March 2025.
[7] T+T (2025) *Pakowhai Secondary Stopbank – Flood Protection Optioneering,* dated 30 May 2025.
[8] T+T (2025) *Consequential Flood Effects – Pakowhai Stopbank,* dated 19 August 2025.
[9] T+T (2025) *Pakowhai Secondary Stopbank – Preliminary Design Report*. Dated 11 September 2025.

  - Seepage based on the current design flood model (200 m^3/s inflow from the Tutaekuri river).
  - A preliminary concept assessment of proposed retaining structures.
- Assessment of possible borrow source options.
- Preparation of this geotechnical assessment report.

2 Background

The Hawke's Bay Regional Council (HBRC) funding criteria aims to protect against a 1-in-100-year flood event. The primary vulnerability for the Pakowhai area is the potential for a catastrophic breach or overtopping of the upstream stopbanks of the major Tutaekuri and Ngaruroro Rivers into the Tutaekuri Waimate (Tk-W) catchment.

In response to this specific risk, HBRC has opted to design a scheme to manage a large inflow of 200 m^3/s into the catchment. This figure was determined as a reasonably achievable and practical level of protection, based on constructing a stopbank 2-3 meters high within local property constraints along the true left of the Tutaekuri Waimate (Tk-W) stream. The intention of this scheme is to act as a secondary stopbank mechanism to reduce the likelihood of flooding into the Pakowhai Category 2 Land areas.

2.1 Site description

The proposed secondary stopbank is located along the true left of the Tutaekuri Waimate (Tk-W) stream in Pakowhai, Hastings District. The alignment is approximately 8 km in length running from Links Road to Hodgson Road through various land parcels, which are generally currently used for agricultural purposes. The alignment itself is split into two by State Highway 2, which runs north to south. The scope of stopbank works is variable along its entire alignment due to variable ground heights (relative to the design levels) and pre-existing (disrepaired) stopbanks. The proposed stopbank works range from stopbank modifications (crest raise and widening) to a full-scale rebuild of a new stopbank.

There are two areas along the alignment where a flood wall or retaining structure may be required due to neighbouring boundary conditions limiting the extent of works. These areas include the FEL block orchard (accessible via 71 Franklin Road) and the structures (garage/shed) at 24 Chesterhope Road.

The entire alignment lies in the centre of the Heretaunga Plains, which is generally flat, with a gradual decrease in existing ground level from approximately 10 to 6 m RL (NZVD16) from northwest to southeast. The Tk-W stream generally sits 2.5 m below the surrounding ground elevation with a maximum channel width of approximately 10 m.

The location of the stopbank and the current proposed alignment is shown on our ground investigation plan shown in Appendix A.

2.2 Published geology and faults

The published geological map of the area[10] indicates that the site is underlain by Holocene River deposits, including silts, sands, and gravels of the Tollemache member of the Heretaunga Formation. The location of the site in the context of the regional geology is presented on Figure 2.1 below.

The GNS active faults database[11] identifies numerous southwest to northeast trending active faults across the western ranges and southern Hawkes Bay region, approximately 10+ km south of the site. The Hikurangi subduction zone, offshore of Hawkes Bay, presents a more significant seismic hazard

[10] Lee, J.M.; Bland, K.J.; Townsend, D.B.; Kamp, P.J.J. (compilers) 2011: Geology of the Hawke's Bay area. Institute of Geological & Nuclear Sciences 1:250,000 geological map 8. 1 sheet + 93 p. Lower Hutt, New Zealand. GNS Science.
[11] Langridge, R.M., Ries, W.F., Litchfield, N.J., Villamor, P., Van Dissen, R.J., Barrell, D.J.A., Rattenbury, M.S., Heron, D.W., Haubrock, S., Townsend, D.B., Lee, J.M., Berryman, K.R., Nicol, A., Cox, S.C., Stirling, M.W. (2016). The New Zealand Active Faults Database. New Zealand Journal of Geology and Geophysics 59: 86-96. https://doi.org/10.1080/00288306.2015.1112818.

to the site. This subduction zone is currently subject to ongoing seismic research. The location of the site in the context of the regional seismicity is presented on Figure 2.3 below.

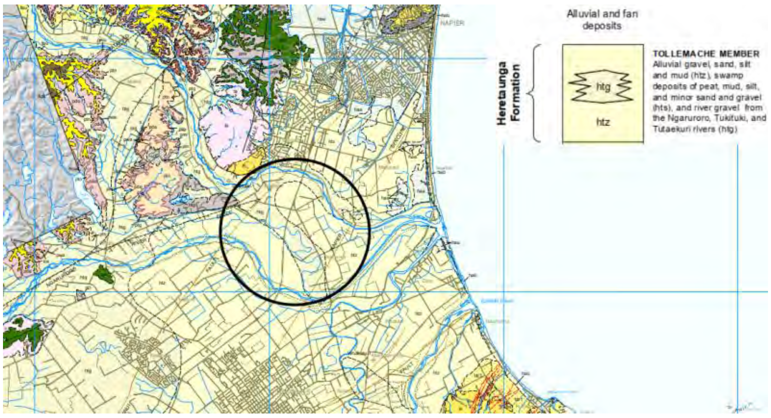

Figure 2.1: Geological setting (general site location circled in black)[10].

Figure 2.2: Active faults (faults indicated in red, general site location circled in black)[11].

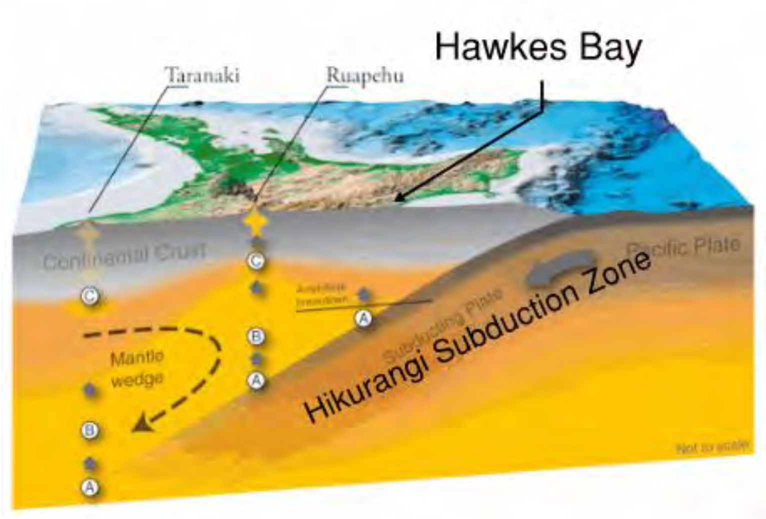

Figure 2.3: The geometry of the Hikurangi subduction zone, which drastically affects the seismic hazard in Hawkes Bay[12].

[12] Geology. (n.d.). Hikurangi Threat. Retrieved September 5, 2025, from https://hikurangisubductionzonenz.weebly.com/geology.ht

3 Site investigations

3.1 Ground investigations

An initial round of geotechnical investigations was carried out by Geotechnics and Geotech Drilling Ltd. across the proposed stopbank alignment and surrounds in August 2024 and December 2024. The investigations comprised:

- 27 No. test pits (TP):
  - Six of which had an adjacent scala tests.
  - The remaining 21 No. had shear vane testing undertaken at 0.5 m intervals in the sidewall of the test pit.
- 14 No. machine boreholes (BH).
- 35 No. hand augured boreholes with an adjacent scala penetrometer test (HA).
- 52 No. cone penetrometer tests (CPT) (ranging in depth from 4.5 metres to 21.3 metres depth).

Investigation locations were selected by T+T on the basis of access, presence of overhead and buried services and traffic management considerations.

Additional ground investigations were completed in July and August 2025 by Geotechnics in areas that had a significant design/alignment change since the original concept design[13]. The ground investigations, completed by Geotechnics and Geotech Drilling, comprise the following additional tests:

- 34 No. test pits (TP).
- 10 No. hand augured boreholes with an adjacent scala penetrometer test (HA).
- 15 No. cone penetrometer tests (CPT).
- Additional laboratory testing to characterize potential fill material.

We note at the time of reporting, these investigations are in progress and will be incorporated into detailed design. However, we have shown these indicative investigation locations on our site investigation plan in Appendix A.

3.2 Groundwater

Groundwater levels were measured from installed borehole piezometers constructed as part of our ground investigations. Groundwater measurements using hand-held dip metres have been undertaken on six occasions since installation in December 2024. The installation details of these boreholes and their geotechnical zone and separable portion are outlined in Table 3.1.

Table 3.1: A summary of the borehole monitoring locations with their separable portion zoning

Borehole ID	Geotechnical Zone	Collar RL (m)	Depth drilled (m)	Installation depth (m)
BH403	GZ – 02	6.91	15.45	3 – 6
BH404	GZ – 05	7.27	19.95	4 – 8
BH409	GZ – 02	8.01	12.45	2 – 5
BH414	GZ – 01	10.15	15.45	1 – 5

[13] T+T(2024). *Pakowhai Concept Drawing Alignment*, issued 10 December 2024, T+T project ref. 1017353.2403.

Groundwater readings from the bore are shown in Figure 3.1. At the time of our investigation the measured groundwater level ranged between 3.5 mRL (lower reaches near the Ngaruroro) to 8 mRL (near Links Road).

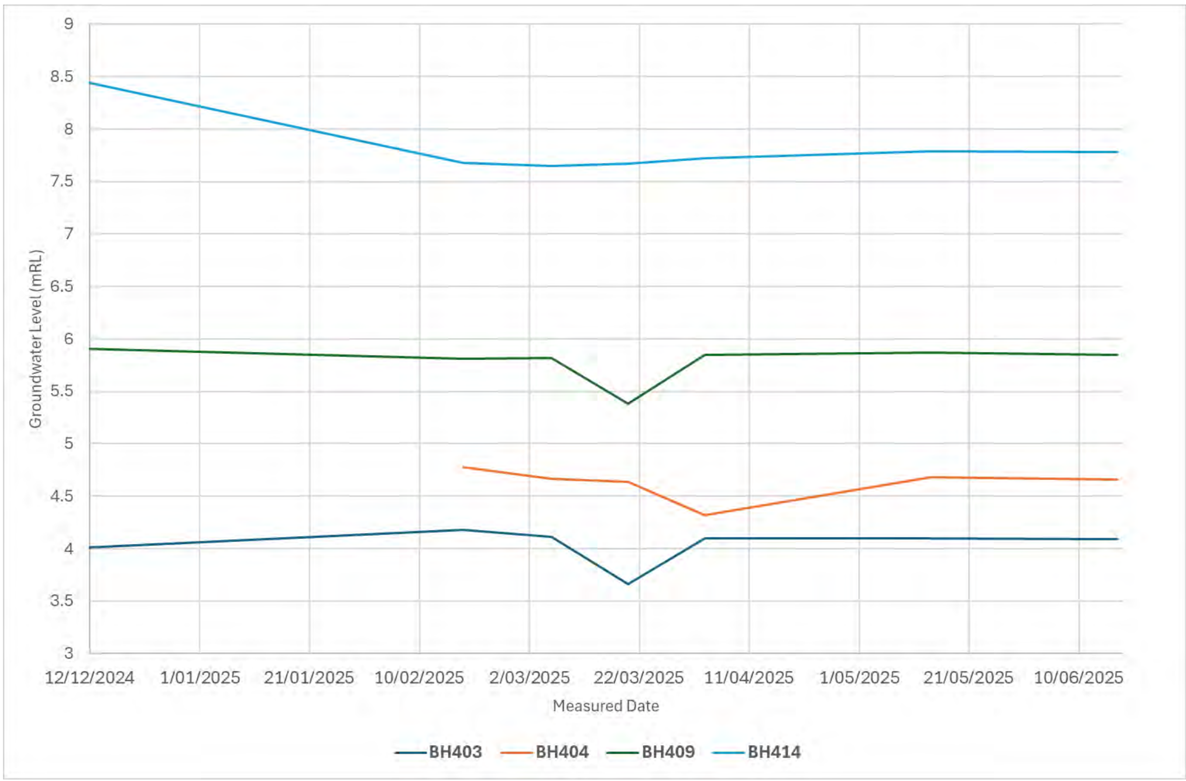

Figure 3.1: Groundwater measurements at each of the borehole piezometer locations.

3.3 Groundwater fluctuations

The nearest groundwater monitoring well (Well 17482), located along the Pakowhai alignment at the corner of Franklin Road (Stopbank 3B4C CH850), provides historical groundwater data available on the Hawke's Bay Regional Council website[14]. We note the nearest piezometer to this monitoring well installed as part of our investigations are at BH409 (1650 m downstream) and BH414 (1000 m upstream). These data indicate typical groundwater level fluctuations of approximately ±0.6 m over the period July 2024 to July 2025. We note the land surface elevation at this location is approximately 10 mRL based on the most recent lidar. Since our investigations were conducted in August and December 2024, the measurements from August are prioritised in our analysis.

We note this period (August 2024) was not necessarily a wet period of year. Therefore, during times of excessive rainfall, the groundwater may be high than what we have measured. The observed rainfall at the Tutaekuri-Waimate stream is shown in Figure 3.2.

[14] Hawkes Bay Regional Council – Environmental Data – Groundwater database. Retrieved 8 April 2025 from https://www.hbrc.govt.nz/environment/environmental-data/groundwater/.

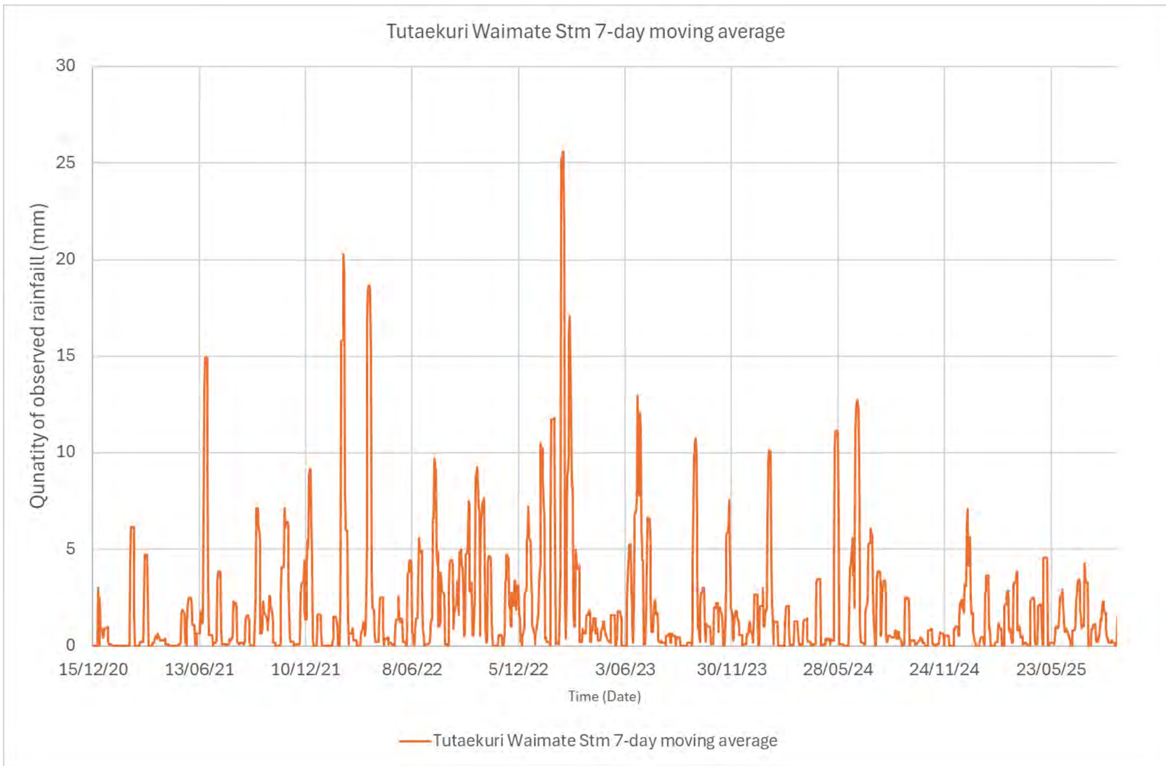

Figure 3.2: The observed rainfall at the Tutaekuri Waimate Stream at Chesterhope weather station.[15]

Figure 4.1 presents the groundwater monitoring data from Well 17482, illustrating groundwater level fluctuations over the past year.

Figure 4.1. One year groundwater monitoring data at Well 17482.

[15] Hawke's Bay Regional Council. (2025). *Rainfall* [Environmental data portal; site: Tutaekuri Waimate Stream at Chesterhope]. Retrieved September 16, 2025, from https://www.hbrc.govt.nz/environment/environmental-data/rainfall/

4 Geotechnical Models

The alignment is underlain by a mix of Holocene alluvial sediments and man-made fill material, varying in strength and composition across the site extent. To establish a ground model and capture ground variability, several geological long-sections were developed by splitting the stop-bank alignment into 10 nominal sections approximately 1000 m in length. The location and information on the geological sections are presented within Appendix A. Interpretation of the geological units feeds into the geotechnical zoning below.

An outline of the key geological units is given below based on the available investigation information.

4.1 Geotechnical zoning (earthen stopbanks)

Due to the large interpolation distances and the significant alignment length, we've developed five geotechnical zones to broadly reflect similar ground conditions. This has been done to simplify the ground-models for assessment purposes, and more readily undertake sensitivity analyses for inputs that affect the design. Each of these zones described below has slightly differing geological profile due to widespread braiding of historical channels of the Ngaruroro and Tutaekuri Rivers.

We note the actual delineation points between zones is likely to vary due to the extensive alignment length and inference between large distances between investigation points. Due to the general nature and attempt at simplicity, the geotechnical zoning is limited to areas where an earthen stopbank is proposed. Areas where a specific structure is proposed as outlined in Section 6.1 and 6.2 are subject to a specific geotechnical model.

A summary of the geotechnical zones can be found in Table 4.1 with the corresponding long sections (found in Appendix A) showing the zonings described below. We note these generalised geotechnical zonings do not apply to areas requiring a specific structure (e.g. hybrid flood wall at the FEL block and retaining wall at 24 Chesterhope Rd., described further in Section 6).

Table 4.1: Geotechnical Zoning Summary

Geotechnical Zone (GZ)	Associated Geological Long Section	Stopbank ID	Chainages
GZ – 01	Long Section 1.1 Long Section 1.2	3B47a	CH0 to 767
		3B47b	CH0 to 375
		3B4C	CH0 to 620
GZ – 02	Long Section 1.2 Long Section 1.3 Long Section 1.4 Long Section 1.5 Long Section 1.6 Long Section 1.7 Long Section 1.8	3B4C	CH620 to 820
			CH720 to 1770
			CH1720 to 1900
		3B50	CH0 to 790
			CH755 to 1755
			CH1705 to 1805
			CH2380 to 2705
			CH2655 to 3655
			CH4000 to 4510
GZ – 03	Long Section 1.6	3B50	CH 1820 to 2380

Geotechnical Zone (GZ)	Associated Geological Long Section	Stopbank ID	Chainages
	Long Section 1.8		CH 3620 to 3660
			CH 3960 to 4000
GZ – 04	Long Section 1.9	3B50	CH4520 to 5490
	Long Section 1.10		CH5490 to 5929
GZ – 05	Long Section 1.8	3B50	CH3660 to 3960

4.1.1 Ground conditions

Based on the zoning above, we have assessed five cross sections representing critical design for each zone. We have determined the critical case for each section based on depth to highly permeable soils (e.g. sands/gravels), depth to liquefiable layers – alluvial silts and sands, and relatively compressible cohesive soil units.

Table 4.2: Ground model breakdown for each geotechnical zone

Geotechnical Zone (GZ)	Modelled groundwater depth (m)	Geological Unit (see Table 4.3)	Typical Thickness Range (m)
GZ – 01	1.7	Unit 3a (Silty SAND/Sandy SILT)	0.3 to 1.0
		Unit 3b/3d (Sands/Gravels)	3 to 5
		Unit 3c (Silts)	5 to 7
		Unit 3b/3d (Sands/Gravels)	>2
GZ – 02	1.7	Unit 3a/3c (Sandy SILT/SILT)	2 to 3
		Unit 3b/3d (Sands/Gravels)	3 to 7
		Unit 3c (Silts)	4 to 8
		Unit 3a (Silty SAND/Sandy SILT)	>2 m
GZ – 03[1]	0.5	Unit 3a (Silty SAND/Sandy SILT)	2 to 3
		Unit 3e (Soft SILTS/CLAYS)	2 to 3
		Unit 3a/3b (Silty SAND/Sand)	2 to 3
		Unit 3c (Silt)	5 to 8
GZ – 04	2.3	Unit 3a (Silty SAND/Sandy SILT)	2 to 5
		Unit 3b (Sand)	3 to 5
		Unit 3c (Silt)	4 to 8
		Unit 3b/3d (Sands/Gravels)	>2
GZ – 05	0.5	Unit 3a (Silty SAND/Sandy SILT)	0.5 to 1.0
		Unit 3b/3d (Sands/Gravels)	8 to 10
		Unit 3c (Silt)	3 to 4
		Unit 3b/3d (Sands/Gravels)	>2

Note:

CH2080 to CH2180 traverses low-lying ground likely to be very soft near the surface with a high groundwater table

For assessment purposes the critical chainages assessed at each geotechnical zone are as follows:

- Geotechnical Zone 1 Section (GZ-01) at Stopbank 3B47a CH 160.
- Geotechnical Zone 2 Section (GZ-02) at Stopbank 3B50 CH 1610.
- Geotechnical Zone 3 Section (GZ-03) at Stopbank 3B50 CH 2155.
- Geotechnical Zone 4 Section (GZ-04) at Stopbank 3B50 CH 4910.
- Geotechnical Zone 5 Section (GZ-05) at Stopbank 3B50 CH 3715.

4.2 Geotechnical Design Parameters

Due to the large alignment and consistent alluvial floodplain geology, we have broken down units into their dominant constituent for the purpose of geotechnical design model development.

Following on from a review of the geological long-sections, dominant lithologies (e.g. sand-dominated, silt-dominated, etc.) have been characterised in Table 4.3. The units vary in thickness and order between various sections assessed. Therefore, we have grouped our sections based on representative critical design cases.

Table 4.3: Geotechnical Design Parameters used in our Seep/w, Slope/w, and Settlement analyses

Unit Ref	Unit	Unit weight (kN/m^3)	Effective Friction angle (°)	Cohesion (kPa)	Constrained Modulus (kPa)*	Hydraulic conductivity, Kh (m/s)**
1	Proposed Stopbank Fill Material	16 *(critical gradient = 0.7)*	32	4	N/A	1×10^{-6}
2	Historic Stopbank Fill (Blended Gravel)	18	34	0	20,000	1×10^{-4} to 1×10^{-5} (2.5×10^{-4})
3a	L to MD Silty SAND/Sandy SILT	18 *(critical gradient = 0.9*	31	2	20,000	1×10^{-4} to 5×10^{-6} (1×10^{-5})
3b	L to MD SAND	18 *(critical gradient = 0.9)*	32	0	20,000	1×10^{-3} to 5×10^{-5} (1×10^{-4})
3c	FIRM SILT	16	28	2	10,000	2×10^{-5} to 1×10^{-8} (1×10^{-7})
3d	MD to D GRAVEL	18 *(critical gradient = 1.0)*	36	0	25,000	1×10^{-2} to 1×10^{-4} (1×10^{-3})
3e	Soft SILTS/CLAYS	15	24	2	5,000	2×10^{-5} to 1×10^{-8} (1×10^{-7})

Note:

*1-D Constrained modulus derived from a combination of Table 11.7 Elastic parameters of various soils, Burt Look (2007) Geotechnical Investigation and Design Tables and the Mayne (2007) CPT correlations and taking into account high variability across the large area.

**Hydraulic conductivity sourced from the CPT-correlation derivation as noted by Robertson and Cabal (2022) and particle size distribution-correlations. Values in brackets assumed for analysis.

***Material order and thickness varies between the sections assessed.

**** Critical gradient is calculated using the following formula and is assessed at the toe heave calculation below

$$Critical\ gradient = \frac{\gamma_{sat} - \gamma_w}{\gamma_w}.$$

5 Geotechnical Assessment

5.1 HBRC performance criteria

The design of the Pakowhai Secondary Stopbank is intended to be consistent with the HBRC consultant briefing documents. As per previous discussions with HBRC and in accordance with the latest HBRC requirements, we have assumed a design life of 50 years, noting the HBRC consultant design brief states a design life of 75 years.

The following performance criteria (sourced directly from the HBRC consultant brief[16]) shall be met:

- HBRC's ideal service level is to withstand a 1/250-year seismic event with no deformation, excluding deformation anticipated from the static case. Deformation during a 1/500-year event vertical and lateral displacement must be within specified limits.
- ULS seismic vertical settlement greater than 200 mm or lateral spread over 500 mm is considered excessive deformation. If such deformation occurs in a ULS event, the corresponding SLS event must show no deformation to be acceptable.
- The maximum vertical exit hydraulic gradient at the toe of the stopbank (I max) is to be 0.5.
- A minimum factor of safety of 1.2 must be achieved against landward side heave during a 0.2% AEP flood event.
- The analytical review must address key issues including sliding failure, overturning failure, slope failures (static, seepage, rapid drawdown), foundation failures (seepage, liquefaction), ground heave, and embankment failure (global stability) from seismic loads.

5.2 Seismic hazard

5.2.1 Ground motion properties

The seismic design parameters have been defined by HBRC in the amendment brief referenced above with the corresponding ground motion parameters presented in Table 5.1.

Table 5.1: Ground Motion parameters defined from HBRC (sourced from MBIE Module 1[17])

Event	PGA (g)	Effective Magnitude (M_w)	Return Period (years)
SLS event (1/25)	0.12	6.4	25
Intermediate event (1/100)	0.26	6.7	100
Intermediate event (1/250)	0.42	7.0	250
ULS event (1/500)	0.58	7.1	500

[16] HBRC (2021) IRG Level of Service Upgrade Project, Consultant Briefing Document for Stopbank TK4123R Omarunui Date: 03/09/2021 (Revision 01)

[17] Ministry of Business, Innovation and Employment. (2021, December 7). *Module 1: Overview of the geotechnical guidelines*. Retrieved September 5, 2025

5.3 Liquefaction assessment

5.3.1 Liquefaction potential

As indicated by the HBRC Hazard Portal, the footprint of the Pakowhai Secondary Stopbank is situated in a high liquefaction vulnerability zone[18]. Based on the high groundwater table and shallow soil units that are predominantly granular, as well as the high levels of shaking under a ULS event, we would anticipate liquefaction triggering to occur along most of the alignment, with varying levels of severity depending on the depth to liquefiable layers.

Based on the soil age, plasticity, and density (based on measured cone tip resistance), Unit 3a (Silty Sand), Unit 3b (Sand), Unit 3d (Gravels) are considered susceptible to liquefaction with Ic values typically less than 2.6. We note that these broader units are variable and so susceptibility will be limited to the looser, more granular soils within the layers.

5.3.2 Liquefaction triggering

Liquefaction at the site has been assessed by T+T considering SLS and ULS design-level earthquakes, which correspond to 25-year and 500-year recurrence interval events, respectively. We have included a sensitivity plot within the results shown in Appendix B identifying when triggering is expected to occur (e.g. intermediate events (100 – yr)).

The liquefaction triggering analyses have been carried out using the methodologies presented in Boulanger and Idriss (2014), with corresponding liquefaction-related consolidation settlements calculated using the method of Zhang et al (2002). Liquefaction severity number (LSN) has also been used to indicate expected liquefaction-induced land damage. Outputs of the analyses plot from T+T’s in-house software are presented in Appendix B.

The performance thresholds for these liquefaction vulnerability indicators are provided in Table 5.2.

Table 5.2: Summary of liquefaction vulnerability indicators (MBIE, 2016)

Expected liquefaction vulnerability	Liquefaction Severity Number, LSN	1D post-liquefaction reconsolidation settlement, S_{v1d}	Non-liquefied crust thickness, CT
None to minor	0 – 16	0 – 25 mm	> 3 m
Moderate	16 – 25	25 – 100 mm	2 – 3 m
Severe	> 25	> 100 mm	< 2 m

5.3.3 Liquefaction assessment summary

A summary of the liquefaction results for each CPT can be found in Appendix B with the full calculation output in the subsequent pages. We have also presented our ULS liquefaction triggering for a 15% Probability of Liquefaction on our long sections shown in Appendix A. We note the estimated settlements are ‘free field’ only and are based on existing ground levels. Therefore, the actual displacements at the surface of the finished stopbank may be different than what is shown within our calculations. We note liquefaction induced settlements may be larger than what has been estimated due to phenomena such as soil ejecta, lateral spreading, and reconsolidation.

[18] Rosser BJ, Dellow GD, compilers. 2017. Assessment of liquefaction risk in the Hawke's Bay Volume 1: The liquefaction hazard model. Lower Hutt (NZ): GNS Science. 108 p. (GNS Science consultancy report; 2015/186).

The overall findings of our liquefaction analysis are as follows:

- Minor to moderate liquefaction-related damage is expected to occur in an SLS level event. Liquefaction-related free-field reconsolidation settlement values (Sv1D) are typically between 6 and 15 mm with up to 55 mm in the most extreme case within the upper 15 m of the soil profile. The corresponding LSN values are typically between 1 and 5 with an LSN of 16 in the most extreme case.
- Moderate to severe liquefaction-related damage is expected to occur in a ULS level event. The liquefaction-related free-field reconsolidation settlement values (S_{v1D}) are calculated to be approximately 50 – 150 mm over the upper 15 m of the soil profile. LSN values are calculated to be approximately 11 – 24.
- Most of the liquefaction triggering (and related damage) is triggered at the intermediate 100 year return period event. At this point, a step-change in liquefaction behaviour occurs indicating land damage is at or near the damage expected from a ULS (500-year return period) event.

5.3.4 Lateral Spread Assessment

We have assessed the level of lateral spreading using the methodology by Zhang et al (2004)[19] at a representative CPT along the stopbank. Our assessment assumes a free face height of 2.5 m for the Tk-W stream. CPT425, located at approximately CH3500, was selected for analysis as it is representative of the moderate to severe liquefaction damage assessed across the site. This assessment assumes 6 m thick liquefiable layer at 1.5 m depth. The groundwater depth at this location was adjusted to account for fluctuations, as discussed in Section 3.2. It is also noted the SLS liquefaction induced vertical settlements at CPT425 are estimated to be 19 mm. we note this is in excess of HBRC's requirements.

The results indicate lateral spreading adjacent to the free face is likely to be in the order of metres under a ULS and intermediate events (100- and 250-year events).

5.3.5 Consequences

The total estimated vertical settlements associated with liquefaction deformation (estimated to be 50 to 150 mm under an intermediate and ULS event) are within HBRC's guidelines.

The assessed global lateral spreading estimate is unlikely to achieve HBRC's lateral displacement requirements (500 mm) until upwards of 30 metres from the crest of the free face under an intermediate (100 year and 250year) and ULS event. We note there are many areas along the alignment where this is not achievable due to property constraints and therefore may not achieve the performance criteria without ground improvement. It is also noted that SLS liquefaction-induced settlements do occur along the alignment and may be subject to minor lateral spread movements. It is noted this is in excess of HBRC's requirements.

Given the stopbank itself can have additional fill placed to bring the flood protection back to the design level following free-field settlements during a seismic event, the primary concern are areas susceptible to near-surface liquefaction with a free-face (e.g. river bank), which could result in lateral spreading into the adjacent stream. This may require a complete rebuild of the damaged stopbanks following a seismic event. An example of this damage is shown in Figure 5.1.

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[19] Zhang, G., Robertson, P. K., & Brachman, R. W. I. (2004). Estimating Liquefaction-Induced Lateral Displacements Using the Standard Penetration Test or Cone Penetration Test. Journal of Geotechnical and Geoenvironmental Engineering, 130(8), 861–871. DOI:10.1061/(ASCE)1090-0241(2004)130:8(861)

Figure 5.1: A photo showing lateral spreading along a flood protection system[20].

Areas where our preliminary liquefaction modelling shows this is could occur and lead to substantial damage (excessive lateral spreading) under an intermediate/ULS seismic case are:

- Stopbank 3B4C:
  - CH1470 to 1770 m (Long Section 1.3).
- Stopbank 3B50:
  - CH 1000 to 1100 m (Long Section 1.5).
  - CH 1800 to 1850 m (Long Section 1.6).
  - CH 2100 to 2200 m (Long Section 1.6).
  - CH 2750 to 4000 m (Long Section 1.7 and 1.8.
  - CH 4700 to 4900 m (Long Section 1.9).

Note, due to large distance between interpreted Cone Penetration testing, lateral spreading damage could occur in areas outside of what has been listed above.

A comprehensive, zone-specific lateral spread assessment should be undertaken during detailed design to confirm this for each separable portion. This will provide an indication for expected damage and possible ground improvement options (if deemed appropriate).

5.4 Seepage and slope stability

A slope stability and seepage analysis were undertaken using HBRC guidelines[11] to understand the risk during long term static, elevated river flows and seismic conditions. We've provided the results along with the HBRC acceptance criteria in Table 5.3 below.

[20] *Sarah Bastin*, via RNZ — **Our Changing World**, "Liquefaction: lessons from the 2011 Christchurch earthquakes," 11 Feb 2021.

5.4.1 Design method

The slope stability and seepage analyses were undertaken using 2D Limit Equilibrium Methods using "Slope/W" and "Seep/W" by GeoStudio.

The Slope/W analyses utilised:

- Morgenstern-Price type analysis.
- Optimisation of slip surfaces.
- Entry and exit and block specified slip surface options to determine the most likely failure.

The Seep/W analyses utilised:

- Saturated/unsaturated type model.
- Seep/W in-built estimation sample functions for volumetric water content and hydraulic conductivity.

5.4.2 Analyses Assumptions

The following assumptions apply to our slope stability and seepage modelling:

1. Soil material properties used in our analysis are summarised in Table 4.3.
2. A traffic loading of 12 kPa has been assessed assuming a lightweight tractor may drive along the crest of the stopbank.
3. Circular failures have been modelled for static, seismic, and flood conditions. The factor of safety (FoS) values presented in Table 5.3 below correspond to the most critical circular failure.
4. Critical slip surface optimisation was turned on for all analyses.
5. Newmark sliding block analysis has been used to determine slope movement for seismic cases with FoS < 1.0. Two methods were used to calculate the displacement under seismic loading:
   - Ambraseys & Srubulov (1995), Jibson (2007) and Martin & Qiu for the rigid block method.
6. For the seepage analysis, floodwater levels and hydrographs were provided by our flood modelling team using the design event (200 m^3/s inflow from the Tutaekuri river with a 0.5% AEP river flood level in the Ngaruroro Overflow channels). The results from this modelling can be found in T+T's flood effects report[8]. The steady state water level was based on the peak flood level from the provided hydrograph.
7. It is unlikely an earthquake will occur at the same time as a large flood event. However, as required by the IRG Consultant brief, we have assessed an SLS earthquake case during the steady state flood as a sensitivity check. We note, HBRC will need to maintain the operability of the stopbank post a seismic event.
8. No boundary conditions were applied vertically at each end of the models. Instead, the models were extended either side of the stopbank to a distance where basin effects at the edge of the models were limited.

5.4.3 Stability and seepage results

Slope/W and Seep/W outputs are presented in Appendix C, and results are summarised in Table 5.3 below.

Table 5.3: Slope stability and seepage results summary for Cross section 1, 2, 3, 4, and 5

Case	Scenario	Target FoS and Criteria	Section 1 (with drain) Stopbank 3B47a, CH160 (GZ - 01)	Section 2 Stopbank 3B50, CH 1610 m (GZ - 02)	Section 3 Stopbank 3B50, CH 2155 m (GZ - 03)	Section 4 Stopbank 3B50, CH 4910 m (GZ - 04)	Section 5 Stopbank 3B50, CH3680 (GZ - 05)
Inputs							
	Maximum flood level (m RL)	N/A	11.35 (+0.35 m above modelled flood level at this chainage)	8.35 (+0.3 m above modelled flood level at this chainage)	8.22 (+0.2 m above modelled flood level at this chainage)	7.85 (+0.1 m above modelled flood level at this chainage)	7.72
	Flood height above stopbank toe (m)		1.9	1.8	2.68	1.6	3.72
	Stopbank toe width		6	5	8	5	10.3
	Flood height/toe width (approx. gradient)		0.33	0.4	0.33	0.33	0.36
	Slope batters		1V:2.5				
	Thickness of surficial silt material		0.5	2.1	2.3	2.7	0.5
	Material below surficial silts		Unit 3d:Gravels	Unit 3d: Gravels	Unit 3b: Sands	Unit 3b:Sands	Unit 3b:Sands
Results							
1a	Static Long Term: River side	FoS > 1.5	2.58	2.66	2.04	2.73	1.91
1b	Static Long Term: Land side		2.70	2.66	2.82	2.74	2.02
1c	Static Long Term: River side – 12kPa Traffic Load		2.21	2.26	1.85	2.29	1.81
1d	Static Long Term: Land side – 12kPa Traffic Load		2.30	2.24	2.41	2.28	1.9
2a	Seismic SLS: River side	FoS > 1.5 or limited displacement (< 100 mm)	1.91	1.97	1.32	2.02	1.37
2b	Seismic SLS: Land side		1.99	1.98	2.09	2.02	1.45
3a	Seismic ILS (100 year): River side		1.45	1.49	0.90	1.52	0.97

Case	Scenario	Target FoS and Criteria	Section 1 (with drain) Stopbank 3B47a, CH160 (GZ - 01)	Section 2 Stopbank 3B50, CH 1610 m (GZ - 02)	Section 3 Stopbank 3B50, CH 2155 m (GZ - 03)	Section 4 Stopbank 3B50, CH 4910 m (GZ - 04)	Section 5 Stopbank 3B50, CH3680 (GZ - 05)
		> 1.2 or displacement less than "damage" limit (< 500 mm horizontal) and < 200 mm vertical)			Estimated displacement < 10 mm		Estimated displacement < 50 mm
3b	Seismic ILS (100 year): Land side		1.50	1.49	1.59	1.52	0.99 Estimated displacement < 50 mm
4a	Seismic ULS: River side		0.90 Estimated displacement < 50 mm	0.92 Estimated displacement < 50 mm	0.47 Estimated displacement < 120 mm	0.94 Estimated displacement < 50 mm	0.57 Estimated displacement < 120 mm
4b	Seismic ULS: Land side		0.93 Estimated displacement < 50 mm	0.93 Estimated displacement < 50 mm	0.98 Estimated displacement < 10 mm	0.94 Estimated displacement < 50 mm	0.56 Estimated displacement < 120 mm
5a	Post - Seismic - Liquefied: River side		2.07	1.58	0.96	1.51	0.66
5b	Post - Seismic - Liquefied: Land side		2.70	2.24	2.82	2.42	0.7
6a	100 year flood using hydrograph supplied by T+T: Riverside (rapid drawdown and seepage under transient conditions)	$FoS_{rapid\ drawdown} > 1.2$	2.46	2.45	2.24	2.76	2.04
6b	100 year flood level with constant seepage: Land side	$FoS_{global} > 1.5$ $I_{max} < 0.5$ $FoS_{toe\ heave} > 1.2$	FoS = 1.9 I_{max} = 0.6 (landward toe) $FoS_{Toe\ Heave}$ = 1.5	FoS = 1.89 I_{max} = 0.4 $FoS_{Toe\ Heave}$ 2.25	FoS = 2.22 I_{max} = 0.1 $FoS_{Toe\ Heave}$ = 9	FoS = 2.03 I_{max} = 0.2 $FoS_{Toe\ Heave}$ = 4.5	FoS = 1.5 I_{max} = 0.6 $FoS_{Toe\ Heave}$ = 1.5

1 Toe heave was calculated by the following: $FoS_{Toe\ heave} = \frac{Critical\ gradient}{Estimate\ gradient}$ $Critical\ gradient = \frac{\gamma_{sat} - \gamma_w}{\gamma_w}$

2 For this specific Imax value, the groundwater level has not reached the ground surface but is close to the surface.

3 Results that do not meet HBRC requirements are highlighted in red.

5.4.4 Stability and seepage Discussion

The static stability criteria along the stopbank alignment generally meets HBRC's requirements. However, there are a couple of cases where the seismic/liquefied criteria are not met. There is also one area where the seepage criteria are not met (the stream diversion).

Hydraulic gradients were assessed across all four cross sections to evaluate the potential for piping on the landward toe during the design flood event. If the hydraulic gradient exceeds the critical hydraulic gradient for a given soil, then the vertical upward forces are expected to exceed the saturated soil weight forces and static-liquefaction, or soil 'boiling' may occur. From this state, soil piping may or may not develop depending on various factors.

The hydraulic gradient exceeding 0.5 along the stream diversion and along the northern alignment nearly exceeding the hydraulic gradients (0.4) are largely driven by shallow groundwater and shallow high permeability soils within the foundations. Where we are nearing the critical gradients, it is recommended to install a landward toe drain to prevent shallow piping leading to instability along the stopbank. At the stream diversion area, it is recommended this follow a more traditional chimney drain given the very shallow groundwater table along the landward side.

5.5 Static Settlement

Silts and clay material are both likely to undergo primary consolidation settlement under the proposed static embankment loads. These settlements are difficult to predict, both with respect to magnitude and time, and are likely to be spatially variable depending on the presence and thickness of softer finer material layers.

Static settlements were analysed in the analysis package Settle 3D for the representative cross-sections analysed within our seepage and stability calculations, assuming a range of 2 to 3.75 metres of fill material (30 to 70kPa stress change). The results of this are presented in Table 5.4.

Table 5.4: Static settlement estimated along the representative cross sections for the zones assessed

Geotechnical Zone	Stopbank/Chainage Ref	Thickness of Stopbank Fill (m)	Thickness of silts (m)	Estimated Settlement (mm)
GZ-01	Stopbank 3B47a, CH160	2.4	0	20 – 30
GZ-02	Stopbank 3B50, CH 1610 m	2.5	2.0	20 – 30
GZ-03*	Stopbank 3B5250, CH 2155 m	3.25	3.0	100 – 150
GZ-04	Stopbank 3B50, CH 4910 m	2.0	2.75	20 – 30
GZ-05	Stopbank 3B50, CH3680	3.75	2.0	40 – 60

*Note, due to the considerably thick **soft** soils and the high fill volumes, excessive settlements are expected for this area. It is recommended that monitoring and overfilling is undertaken to account for this consolidation settlement

Using our design consolidation parameters shown in Table 4.3, we estimate that static settlements under a typical embankment load could be in the order of 20 to 150 mm over a 50-year period noting the range of values above.

We expect that most settlements along the alignment (GZ-01, GZ-02, and GZ-04) will occur relatively quickly (i.e. within a matter of several days to weeks). However, longer-term consolidation settlements in other areas of the alignment (GZ-03 and GZ – 05) are likely to occur well beyond the construction window (months to years) due to the nature of the soft shallow soils. It is recommended the stopbank is over-filled in these locations to accommodate the additional significant settlements.

We recommend stopbank levels to be monitored either immediately after construction or as part of a long-term maintenance plan (e.g. an annual drone survey) to ensure the stopbank levels meet the level of service requirements. This is to be confirmed by HBRC at detailed design.

6 Retaining/Flood Wall Design

We note the generalised geotechnical zoning described above does not apply to these site-specific areas. This is due to the increase complexity of these structures, which warrant site-specific geotechnical models.

There are a number of locations along the stopbank alignment, which may require installation of a retaining structure. For each of the locations we have performed a preliminary assessment of a selection of options available. Specific recommendations for each site area have been included within the relevant sections below.

6.1 71 Franklin Road (FEL Kiwifruit Block)

There is a very narrow section of available land available for a flood protection system between Stopbank 3B4C CH910 to CH1710 along the orchard property at 71 Franklin Road. We understand the corridor must be between the existing tensioned ground anchors holding a kiwifruit canopy and the existing river edge. In some cases, this presents significant challenges for an earthwork stopbank solution.

A retaining wall workshop[21] was held with Hawkes Bay on Monday, 28th July where a number of retaining structures were discussed. The outcome of this workshop was to investigate sheet piling options available to HBRC. Initial discussions revolved around use of PVC sheet piling options. However, we understand a number of previously used but good quality STU1800 steel sheet piles have been made available for HBRC to purchase. Therefore, we have focused our preliminary assessment on the use of this steel sheet pile for the area. Figure 6.1 shows the site layout with the areas with a proposed retaining structure.

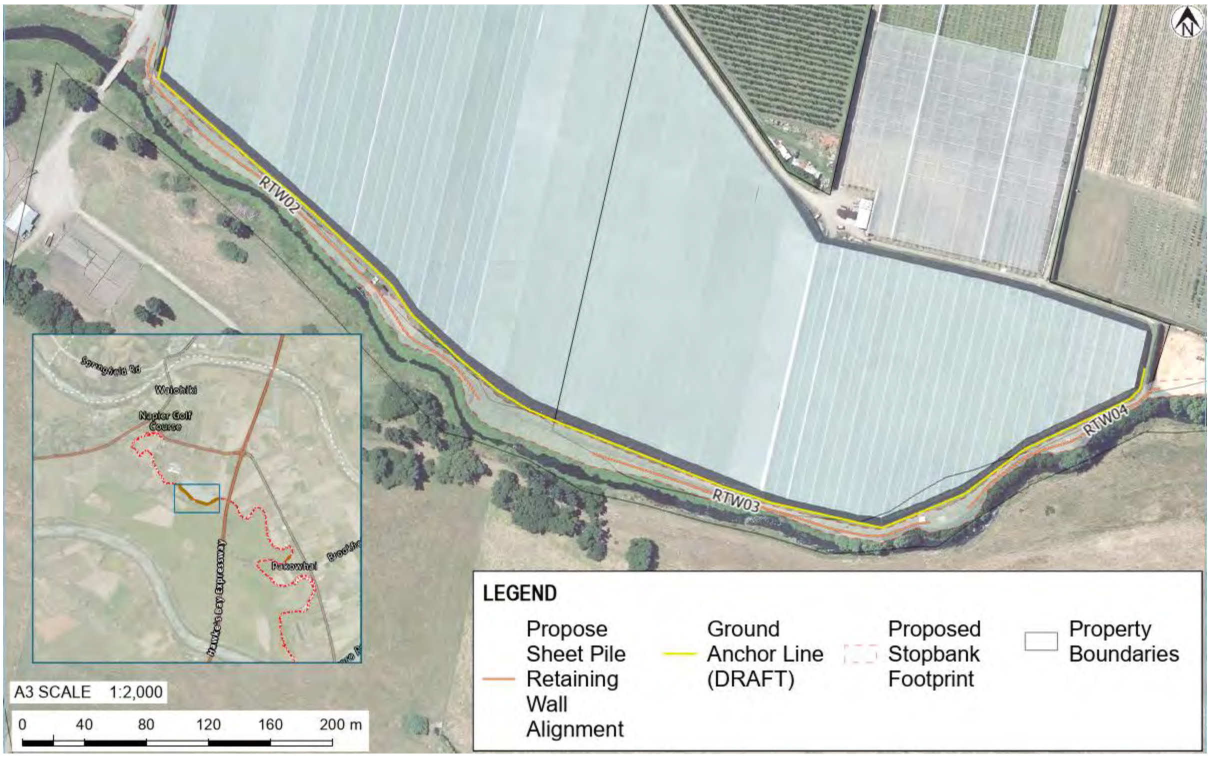

Figure 6.1: The alignment of the proposed sheet piling floodwall option.

[21] Hawke's Bay Regional Council. (2025, July 28). *FEL Block flood retaining structure workshop* [Private meeting]. HBRC offices/online.

To reduce the use of sheet piling, and thus cost, we have assessed a hybrid stopbank-floodwall. A typical section of this is shown in Figure 6.4.

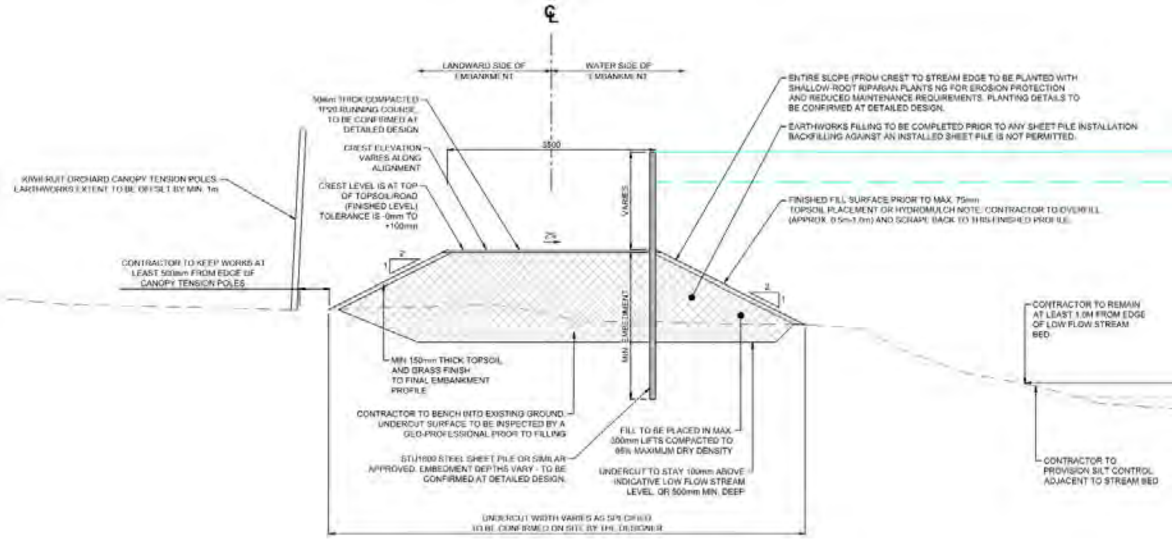

Figure 6.2: The proposed hybrid sheetpile floodwall retaining option.

Using the geotechnical software package, WALLAP[22], we have assessed an STU1800 sheet pile for a single simplified ground model. The ground model used for our analysis assumes near surface soft silts (present around the pump shed and along the alignment) with underlying gravelly sands and an engineered fill cap (stopbank fill). The parameters used in our analysis is shown in Table 6.1 with the depth of each unit.

Table 6.1: Soil Parameters used in our WALLAP and slope/w calculations

Unit	Depth Range	Unit weight, ɣ (m)	Effective cohesion, c' (kPa)	Effective friction angle, ϕ (°)	Youngs Modulus, E (kN/m²)
0. Engineered fill [Silty SAND]	Varies	18	-	34	40,000
1. Soft surficial SILT	0 to 2 metres	17	2	24	4,000
2. Gravelly SAND	> 2 metres	19	-	34	40,000

The result of our WALLAP analysis indicates the following required embedment depth and the expected length that will be required at the specified embedment. We note further analysis is required at detailed design to refine the retained lengths and allow for further detailing of the wall.

[22] Geosolve (2024). WALLAP. Anchored and Cantilevered Retaining Wall Analysis Programme. Version 6.09

Table 6.2: The minimum required embedment with the expected water retained height and expected length required at the sizing based on the proposed stopbank geometry

Max height (Height above stopbank/ ground level) (m)	Minimum embedment (m)	Total Length (m)	Approximated wall length required (m)
2.0	5.0	7.0	415
2.5	6.5	9.0	290

Notes:

1. We have assumed the sheet pile wall will not withstand the loading/effects of lateral spreading and will require specific assessment after a liquefaction-triggering earthquake event.
2. Our pile capacity assumes no slippage between the sheet piles during hydrostatic loading.
3. We have assumed loading from a hydrostatic load and have not taken into consideration impact loading (e.g. from wave action or a tree).
4. We have assumed a 50-year design life and classed the site as "moderately" corrosive as defined by NZS 3404:2018 to determine the corroded pile capacity.
5. 355 MPa graded steel has been assumed for our calculations.
6. We have not performed a scour calculation at the toe of the existing stopbank and have not assessed if this will be eroded based on the modelled flooding.
7. We have assumed all earthworks will be completed, then the sheet piles will be driven through the competent fill material.
8. Any proposed sheet piles that have been previously installed are subject to engineer's approval.
9. Sheet piles shall be installed within the following tolerances: in plan, ±50 mm of the given sheet pile line at commencing surface, vertical 1 in 75, and level -0 and +20 mm of required top level.
10. Caulking between sheet pile joints to be considered at detailed design.

6.2 24 Chesterhope Road

The stopbank alignment within the property at 24 Chesterhope Road is constrained due to several nearby structures. Figure 6.3 shows the site layout with one of the proposed retaining options.

This site area has limited ground investigations, but is understood to have significant soft ground at shallow depths. This soft ground is positioned where the maximum resisting forces are applied for typical retaining solutions. Therefore, a significant retaining structure will be required to meet the flood protection level of service.

Figure 6.3: The layout of the property at 24 Chesterhope Road with one of the retaining wall options assessed.

6.2.1 Riverside kingpost-deadman anchor wall

A preliminary retaining wall design was analysed for this area as part of the proposed secondary stopbank upgrade, assuming a typical stopbank profile with a riverside retaining wall.

We have undertaken a preliminary WALLAP[22] analysis using geotechnical design parameters derived from the nearby CPT434. We have updated soil properties for this retaining wall alignment to reflect a very soft near surface material (the softest amongst GZ-03).A typical cross section of the assessed wall is shown in Figure 6.4.

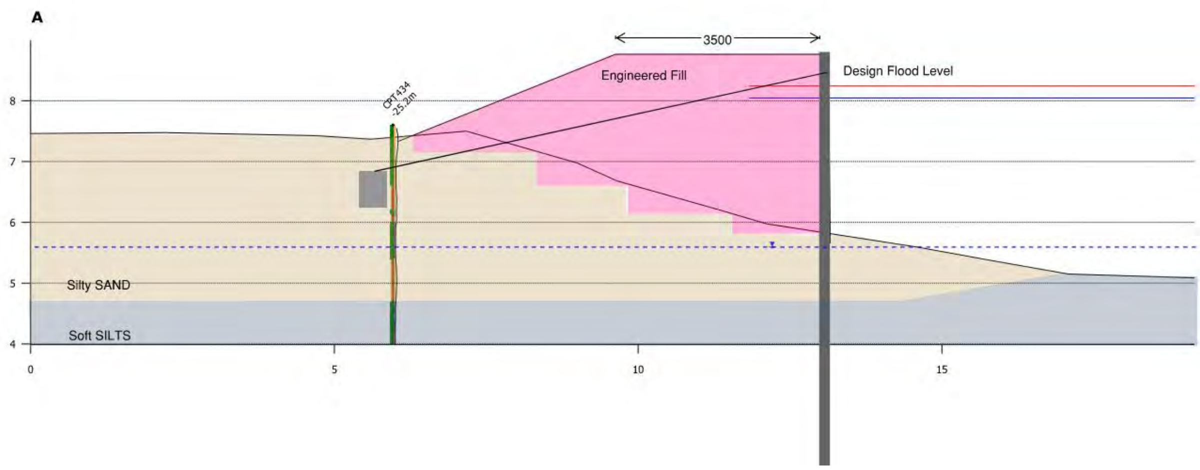

Figure 6.4: A typical cross section of the kingpost with an anchored solution of the proposed retaining option.

We have provided the expected soil design parameters used in the analysis in Table 6.3. The following Wallap cases have been considered as part of our assessment:

- Static loading (associated with fill material).
- Heavy vehicle (earthworks vehicle) surcharge loading (24 kPa).
- A ULS (IL2) earthquake.

Table 6.3: The geotechnical design ground model used in our preliminary WALLAP analysis

Depth below top of proposed retention design (Elevation of top of stratum in brackets)	Unit	Unit weight (kN/m3)	Friction angle (°)	Cohesion (kPa)	Constrained Modulus (kPa)
0 mbgl (8.8 mRL)	Proposed Stopbank Fill Material	16	32	4	20,000
1.4 mbgl (7.4 mRL)	Silty SAND / Sandy SILT	17	32	1	20,000
3.8 mbgl (5 mRL)	Soft CLAY / SILT	16	26	2	2,000
5.3 mbgl (3.5 mRL)	Dense SAND	18	38	0	40,000
9.7 mbgl (-0.9 mRL)	SILT	17	29	2	15,000

The preliminary assessment concludes a 12 m long 254UC89 with RB25 tie-back anchors at 2 metre centres connecting to a deadman block (estimated to be a continuous strip footing at 1.5 m x 1.5 m) could be a suitable retention option, considering the proposed maximum retained height (3.5 metres) and surcharge loading (assumed heavy traffic loading) above the retaining wall.

Section 5.2 outlines the significant seismic loading under a ULS earthquake for IL2 structures. Due to the significant cost associated with resisting IL2 seismic loading, we understand HBRC may accept a reduced importance level (IL1) for this wall. Therefore, based on our preliminary assessment, surcharge loading (as a result of heavy/earthworks vehicle loading) is likely to be the governing case of the design.

If this wall type is chosen to proceed to detailed design, this loading case should be agreed with HBRC.

6.2.2 Riverside ground improvement

An alternative option to be assessed at detailed design is a river-side ground improvement with a landward retaining structure to be constructed around the existing infrastructure. We note this option would likely require a reduced crest width (to 2.0 metres wide) due to the footprint available. Figure 6.5 shows a sketch of the generalised geometry of this preliminary option.

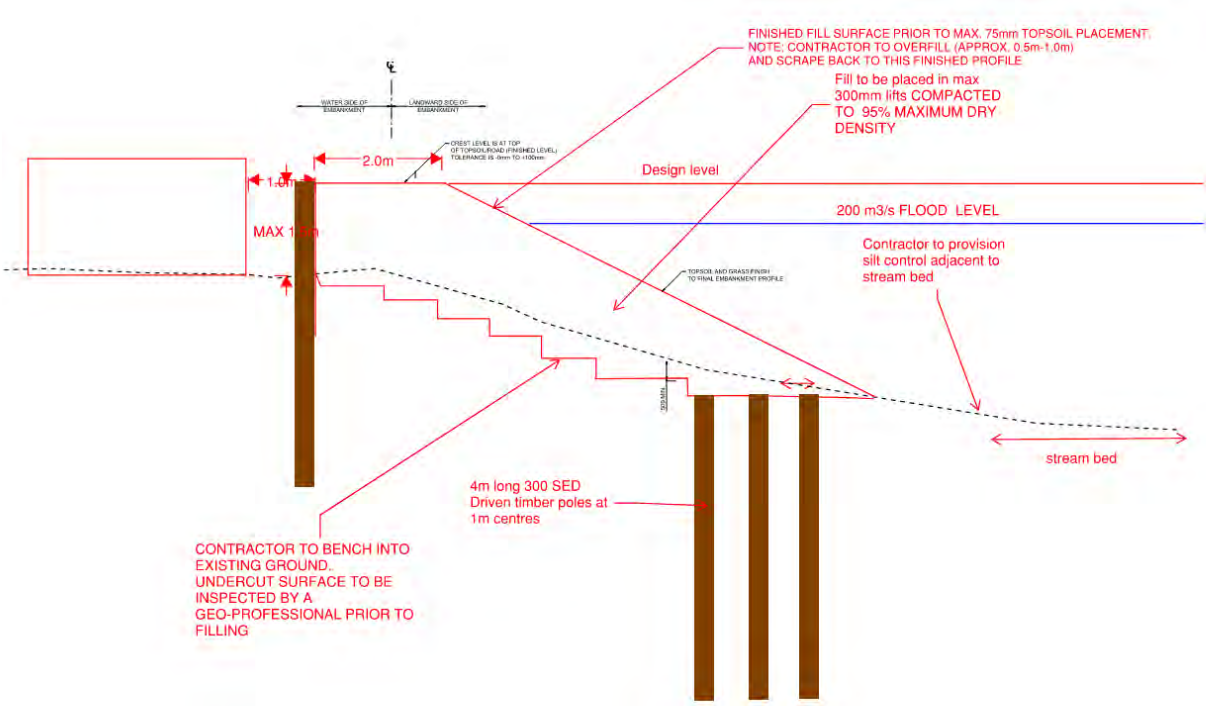

Figure 6.5: An example of a possible ground improvement option (to limit soil mobilisation in the soft soils at the base of the stopbank.

6.2.3 Construction considerations

We have received preliminary pricing from HBRC's contracting consultant, which indicates the ground improvement option could be significantly cheaper to construct. We recommend this option is assessed further at detailed design with HBRC's engineering team to confirm the assessed geometry meets the engineering team's requirements.

7 Seepage Control

To mitigate the seepage risks identified, it is recommended that the stopbank be constructed using (silty sand/sandy silt) or at least incorporate a higher proportion of it to reduce overall permeability. With this material, the transient seepage analysis satisfies the HBRC criteria.

If HBRC intends to meet design criteria under steady-state (constant flood level) conditions as well, additional seepage control measures should be considered to manage elevated hydraulic gradients:

- A stability berm on the landward side would help dissipate excess pore pressures, enhance toe stability, and reduce the potential for heave.
- A drainage trench or relief well could provide a controlled exit path for seepage, thereby reducing gradients through the critical foundation layer.

We recommend that seepage risks and control options to be investigated further during the detailed design phase of the stopbank upgrade works.

8 Geotechnical Considerations

8.1 Borrow fill material

We understand fill material for the Pakowhai secondary stopbanks is likely to vary across the alignment. The southernmost extent of the alignment (CH4680 to CH5924) will likely consist of a silty sand/sandy silt in the upper 2 to 3 metres of surface along the true left bank of the Ngaruroro River.

A geotechnical investigation undertaken by RDCL[23] comprising 37 test pits and associated laboratory testing was undertaken to characterise the borrow site's material suitability. Particle size distribution (PSD) data from this sampling is shown in Figure 8.1 below in **yellow**. This material is generally within the particle size bands as specified by HBRC's fill requirements. T+T have undertaken additional sampling of gravel beaches and the river berm in this area (shown in **green**).

For the remainder of the site, we understand several nearby quarries in the Hawkes Bay are being assessed to provide fill material to construct the stopbank. We have performed an initial assessment of material sourced from the AKM (**blue**) and Matuku Road (**black**) quarries. The plot below shows Ngaruroro River silty sand and AKM quarry materials assessed are within the acceptable particle size distribution band (red) required of stopbank fill.

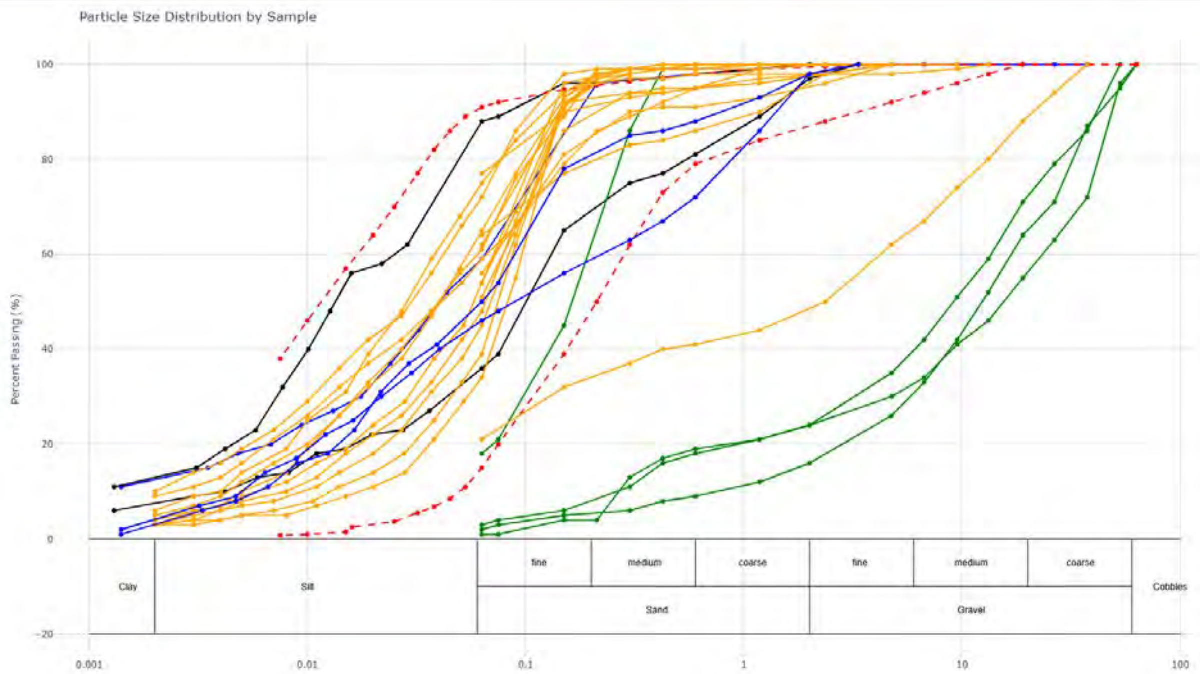

Figure 8.1: A particle size distribution plot of all known borrow sources at the time of writing. The red dashed line shows the HBRC required material for fill placed for stopbanks.

We understand as part of the HBRC consultant brief, proposed fill material with more than 10% passing the 63-micron sieve requires Atterberg Limits testing. HBRC considers that material classified as 'high' for shrinkage and swelling are not suitable. We have observed that some material sourced from Matuku Road is above this acceptable limit. We also note the HBRC consultant brief also requires the Coefficient of Uniformity (D60/D10) for borrow material must be over 4 to ensure sufficient binding and to decrease permeability. Table 8.1 shows the calculated uniformity coefficient averaged by source, noting almost all sources meet HBRC's requirements (with the

[23] RDCL (2025) *Geotechnical Factual Report – HBRC IRG LoS Borrow Catalogue.* Dated 5 June 2025.

exception of four samples from the Ngaruroro Berm). This is due to a D10 not being fully identified under a wet sieve alone (D10 assumed as the smallest sieve size).

Table 8.1: The average coefficient of uniformity by sample source (Derived from particle size distribution)

Material source	Number of lab samples	Coefficient of uniformity (D60/D10)		
		Min	Max	Mean
Ngaruroro River Berm (Silty Sands)	17	1	22.33	7.76
Ngaruroro River (Gravels)	4	21.52	87.41	57.17
AKM Quarry	3	15.66	47.85	36.90
Matuku Road Quarry	2	19.23	32.38	25.80

9 Conclusions

The geotechnical assessment indicates the Pakowhai Secondary Stopbank is generally feasible against HBRC static and transient flood performance criteria across most of the alignment. This is largely due to sufficient space and the groundwater is locally deep enough to avoid shallow seepage (GZ-02 and GZ-04).

However, special consideration is required at the following locations based on our assessments.

Table 9.1: Areas requiring special consideration as part of the stopbank design

Location	Construction consideration
GZ-01 (Northern Alignment)	A landward toe drain, comprising 6 m length and 600 mm thick of clean sand (filter compatible with the stopbank fill material) may be required to accommodate anticipated landward toe seepage during a design event. The extent of this drain should be evaluated at detailed design.
GZ-02 (FEL Block Property)	We note GZ-02 is highly space constrained at 71 Franklin Road. T+T are currently progressing a sheet pile flood wall in this area to achieve the design levels.
GZ-03 (Around Burgess Property):	Significant areas of shallow, soft ground where consolidation settlements could occur may reach up to approximately 150 mm. Over-filling of this area with up to 200 mm of additional material along with settlement monitoring is recommended in this area. 24 Chesterhope Road is highly space constrained due to adjacent shed infrastructure. T+T is progressing optioneering with HBRC to achieve the design requirements with a retaining or ground improvement option.
GZ-05 (River diversion area)	A landward toe drain, comprising 6 m length and 600 mm thick of clean sand (filter compatible with the stopbank fill material) may be required to accommodate anticipated landward toe seepage during a design event. Additional filling may be required to resist upward pore pressures on the landward side of the stopbank.

We have assumed a lower permeability fill (silty sands) for the purpose of preliminary design. However, we note borrow material selection remains to be confirmed during detailed design. The extent of remedial options and flood/retaining structures discussed above are to be confirmed and further evaluated at detailed design.

10 Applicability

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

Recommendations and opinions in this report are based on data from discrete investigation locations. The nature and continuity of subsoil away from these locations are inferred but it must be appreciated that actual conditions could vary from the assumed model.

During excavation and construction, the site should be examined by an engineer or engineering geologist competent to judge whether the exposed subsoils are compatible with the inferred conditions on which the report has been based. We would be pleased to provide this service to you and believe your project would benefit from such continuity. However, it is important that we be contacted if there is any variation in subsoil conditions from those described in the report.

Tonkin & Taylor Ltd
Environmental and Engineering Consultants

Report prepared by:

....

Miles Buob
Geotechnical Engineer

Authorised for Tonkin & Taylor Ltd by:

....

Tim Morris
Project Director

Reviewed by Dan Mills, Senior Geotechnical Engineer

18-Sept-25
\\ttgroup.local\corporate\wellington\tt projects\1017353\1017353.2403\issueddocuments\20250918 geotechnical assessment report\25.09.18_geotechnical assessment report.docx

Appendix A Figures

- **Ground Investigation Plans**
    - **1017353.2403 – SB-F1 to F6**
- **Long Sections (1.1 to 1.10)**
- **Geotechnical Zoning Cross sections**
    - **GZ – 01**
    - **GZ – 02**
    - **GZ – 03**
    - **GZ – 04**
    - **GZ – 05**

C:\Users\mibu\Working\ArcGIS\HBRC_pakowhai_secondary_stopbank\HBRC_pakowhai_secondary_stopbank.aprx Layout: 1017353.2403 - SB-F2 (Investigation Map Series) 2025-Sept-08 9:56 am Drawn by MIBU

LEGEND

- Property Boundaries
- 0.5m interval
- 1m interval

Ground Investigations

- Borehole
- Borehole w/piezo
- Cone Penetrometer Test
- Hand Auger
- Hand Auger w/ piezo
- Test Pit

Geotechnical Zone

- GZ-01
- GZ-02
- GZ-03
- GZ-04
- GZ-05

NOTES

Basemap: NZ Hybrid Reference Layer: Eagle Technology, LINZ, StatsNZ, NIWA, Natural Earth, © OpenStreetMap contributors. NZ Imagery: Eagle Technology, Land Information New Zealand, GEBCO, Community maps contributors

LOCATION PLAN

REV	DESCRIPTION	GIS	CHK	DATE
0	First version	MIBU	TGM	21/03/25

DESIGNED	MIBU	SEPT.25
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CHECKED	TGM	SEPT.25

TGM 21/03/25

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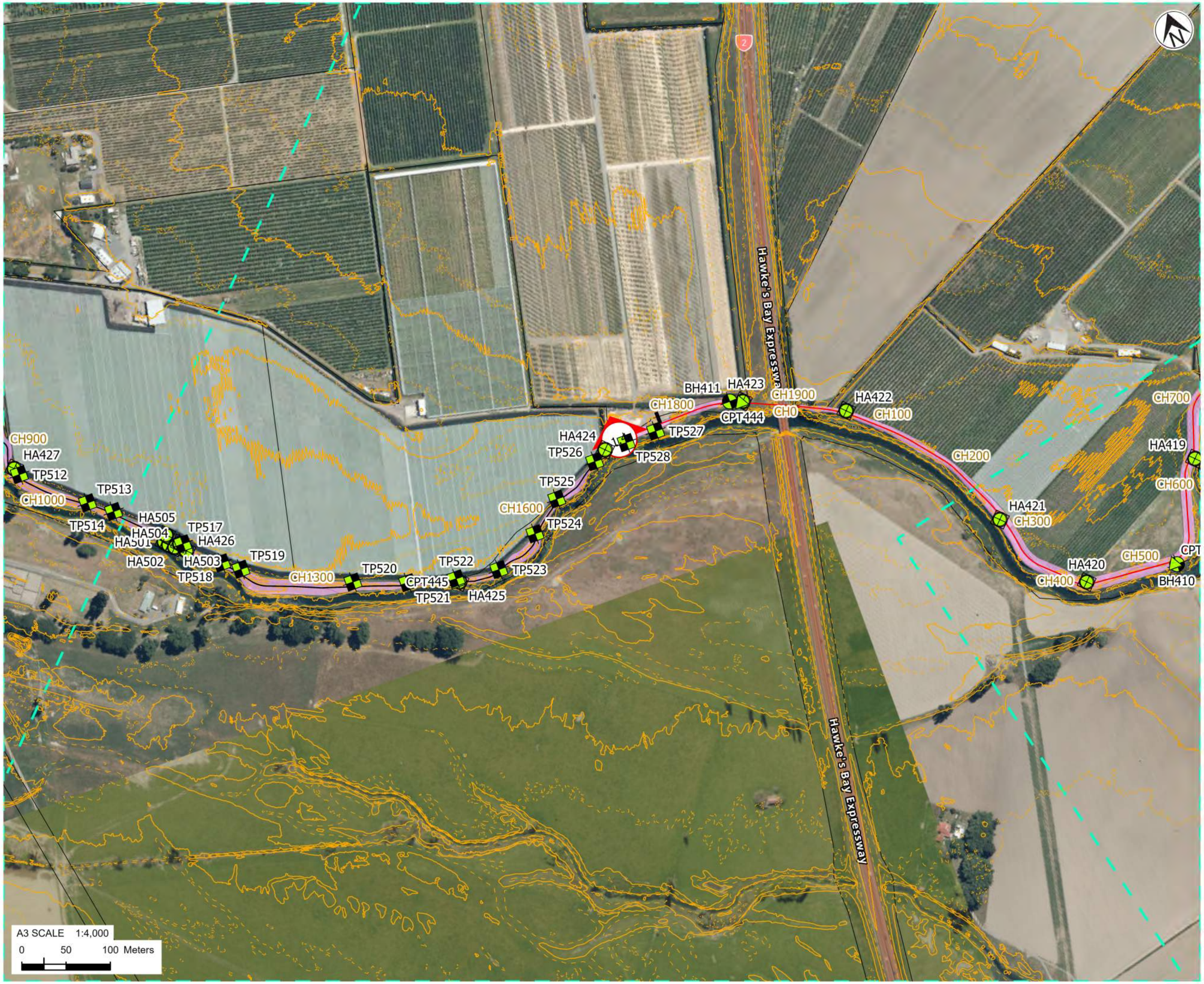
HAWKES BAY REGIONAL COUNCIL

PAKOWHAI SECONDARY STOPBANK

GROUND INVESTIGATION PLAN

PROJECT No. 1017353.2403
FIG No. 1017353.2403 - SB - F2
SCALE (A3) 1:4,000 REV 0



LEGEND

- Property Boundaries
- 0.5m interval
- 1m interval

Ground Investigations

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- Borehole w/piezo
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- Hand Auger
- Hand Auger w/ piezo
- Test Pit

Geotechnical Zone

- GZ-01
- GZ-02
- GZ-03
- GZ-04
- GZ-05

NOTES

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LOCATION PLAN

Pakowhai

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GROUND INVESTIGATION PLAN

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FIG No. 1017353.2403 - SB - F3

SCALE (A3) 1:4,000 REV 0

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LEGEND

Property Boundaries
0.5m interval
1m interval

Ground Investigations
- Borehole
- Borehole w/piezo
- Cone Penetrometer Test
- Hand Auger
- Hand Auger w/ piezo
- Test Pit

Geotechnical Zone
- GZ-01
- GZ-02
- GZ-03
- GZ-04
- GZ-05

NOTES

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HAWKES BAY REGIONAL COUNCIL

PAKOWHAI SECONDARY STOPBANK

GROUND INVESTIGATION PLAN

PROJECT No. 1017353.2403
FIG No. 1017353.2403 - SB - F4
SCALE (A3) 1:4,000 REV 0

LEGEND

- Property Boundaries
- 0.5m interval
- 1m interval

Ground Investigations

- Borehole
- Borehole w/piezo
- Cone Penetrometer Test
- Hand Auger
- Hand Auger w/ piezo
- Test Pit

Geotechnical Zone

- GZ-01
- GZ-02
- GZ-03
- GZ-04
- GZ-05

NOTES

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PAKOWHAI SECONDARY STOPBANK

GROUND INVESTIGATION PLAN

PROJECT No. 1017353.2403

FIG No. 1017353.2403 - SB - F5

SCALE (A3) 1:4,000 REV 0

LEGEND

Property Boundaries

0.5m interval

1m interval

Ground Investigations

Borehole

Borehole w/piezo

Cone Penetrometer Test

Hand Auger

Hand Auger w/ piezo

Test Pit

Geotechnical Zone

GZ-01

GZ-02

GZ-03

GZ-04

GZ-05

NOTES

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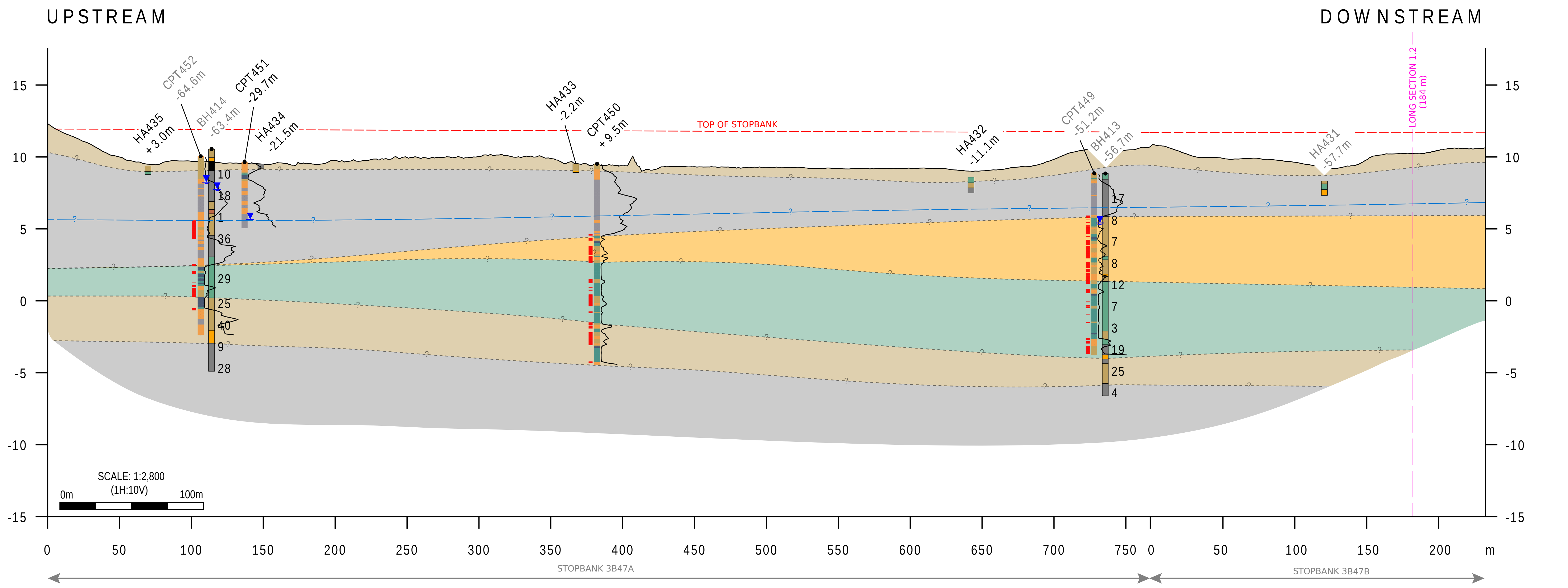
GROUND INVESTIGATION PLAN

PROJECT No. 1017353.2403

FIG No. 1017353.2403 - SB - F6

SCALE (A3) 1:4,000 REV 0



LITHOLOGY

EXISTING STOPBANK FILL (SILTY SAND)

HOLOCENE RIVER DEPOSITS

- CLAY DOMINATED (IC 3 TO 3.6)
- SILT DOMINATED (IC 2.6 TO 3)
- SILTY SAND DOMINATED (IC 2 TO 2.6)
- SAND DOMINATED (IC 1.3 TO 2)
- GRAVEL DOMINATED (IC < 1.3)

LONG SECTION JOIN LINE

BOREHOLE

0 SPT 'N' VALUE

CONE PENETRATION TEST

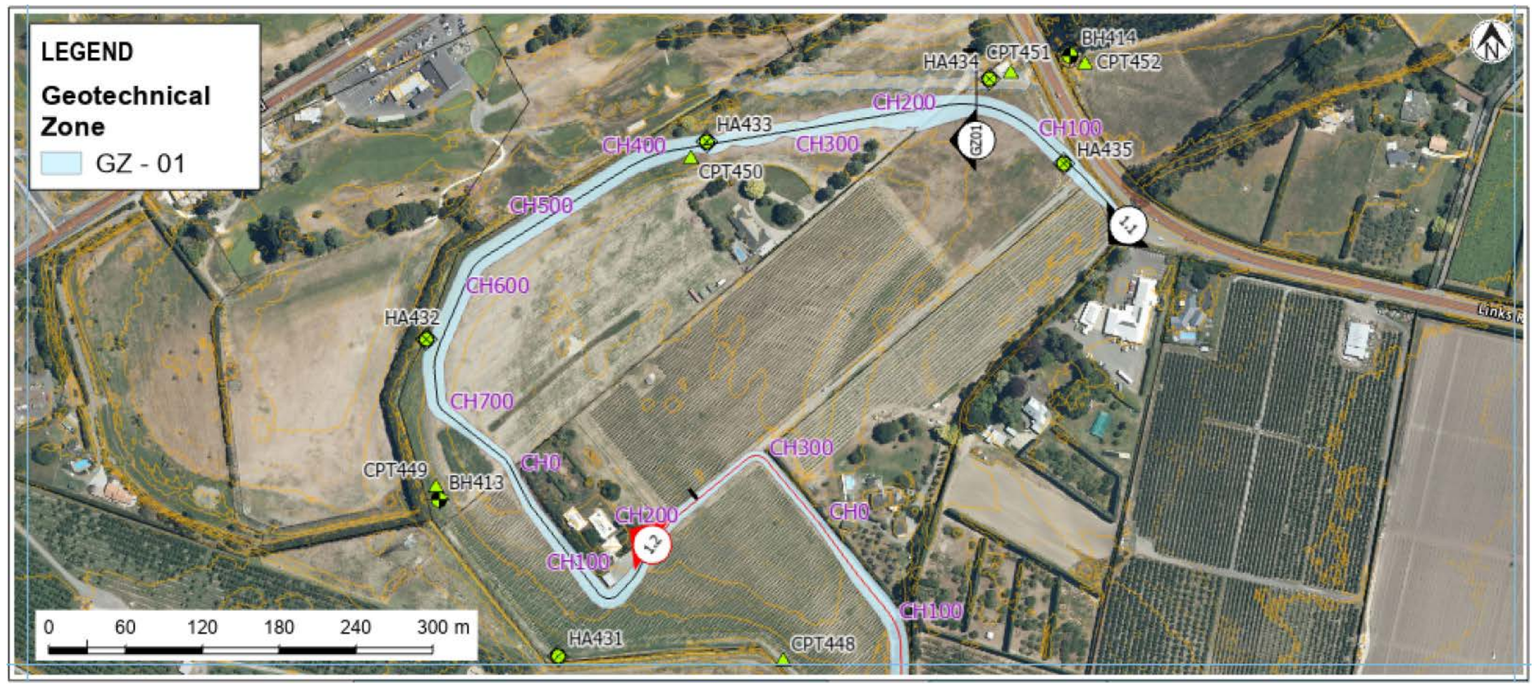
CONE RESISTANCE, qc

LIQUEFIABLE LAYERS (ULS FOS <1)

GROUNDWATER

MEASURED GROUNDWATER LEVEL (DECEMEBER 2024)

INFERRED GROUNDWATER LEVEL



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1. THE CONE PENETRATION TEST SOIL BEHAVIOUR INDEX (IC) HAS BEEN DERIVED USING THE METHODOLOGY DESCRIBED IN ROBERTSON AND WRIDE, 1998.
2. INVESTIGATIONS OVER 40M OFFSET FROM THE SECTION ALIGNMENT HAVE BEEN COLOUR GREY
3. ELEVATION LEVELS ARE WITHIN NZVD16 VERTICAL DATUM
4. WE HAVE GROUPED THE UNITS BASED ON THE DOMINANT GEOLOGICAL UNIT AND WHAT IS LIKELY TO LEAD TO MORE CONSERVATIVE CONDITIONS IN OUR MODELLING

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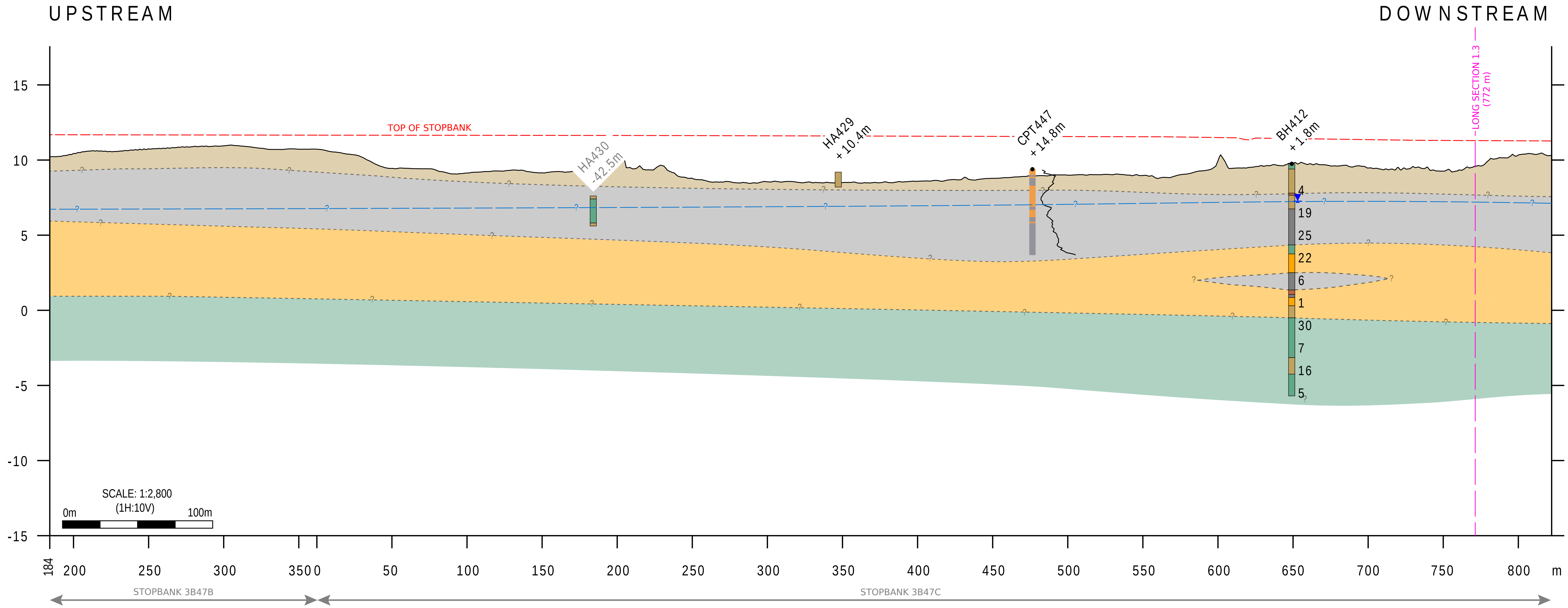
CLIENT **HAWKES BAY REGIONAL COUNCIL**

PROJECT **PAKOWHAI SECONDARY STOPBANK**

TITLE GEOLOGICAL LONG SECTION 1.1

SCALE (A3) AS SHOWN FIG No. 1.1 REV 1



LITHOLOGY

EXISTING STOPBANK FILL (SILTY SAND)

HOLOCENE RIVER DEPOSITS

- CLAY DOMINATED (IC 3 TO 3.6)
- SILT DOMINATED (IC 2.6 TO 3)
- SILTY SAND DOMINATED (IC 2 TO 2.6)
- SAND DOMINATED (IC 1.3 TO 2)
- GRAVEL DOMINATED (IC < 1.3)
- LONG SECTION JOIN LINE

BOREHOLE

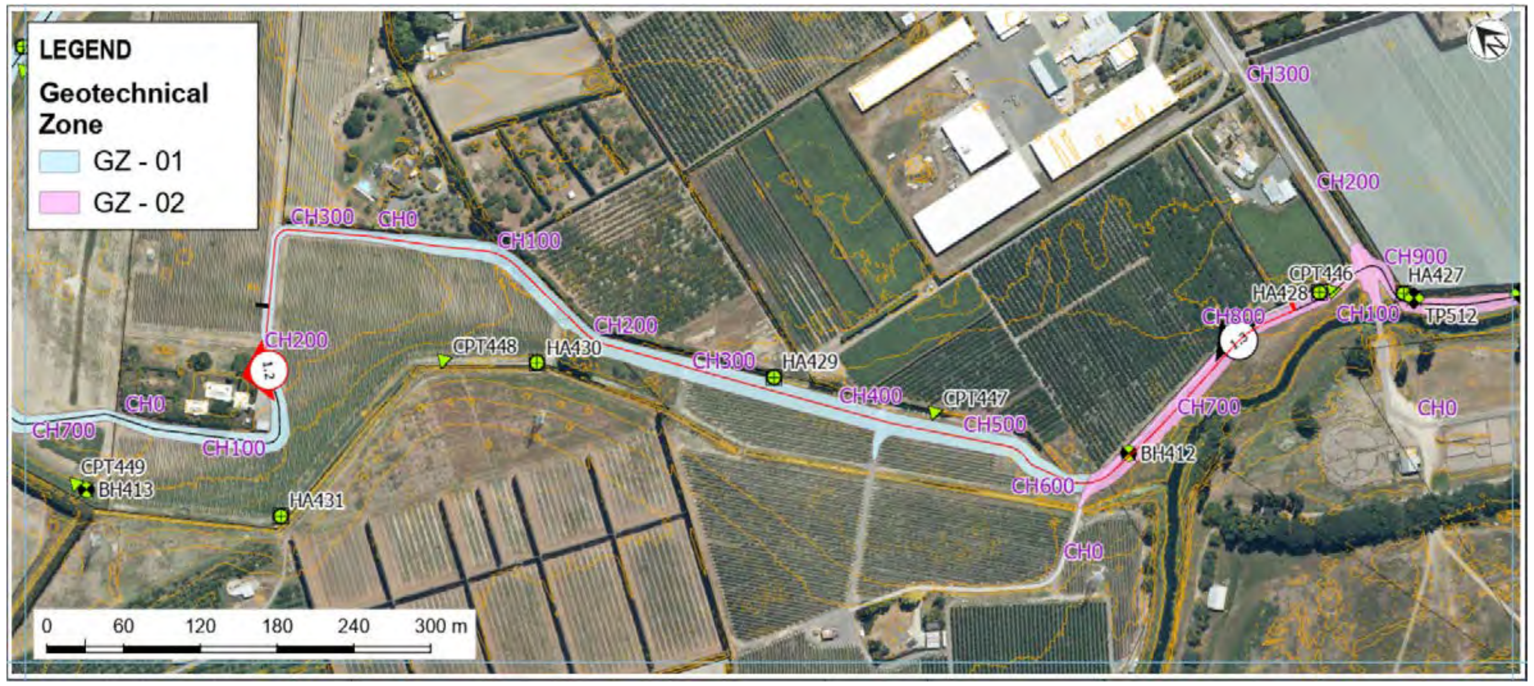
0 SPT 'N' VALUE

CONE PENETRATION TEST

- CONE RESISTANCE, qc
- LIQUEFIABLE LAYERS (ULS FOS <1)

GROUNDWATER

- MEASURED GROUNDWATER LEVEL (DECEMEBER 2024)
- INFERRED GROUNDWATER LEVEL



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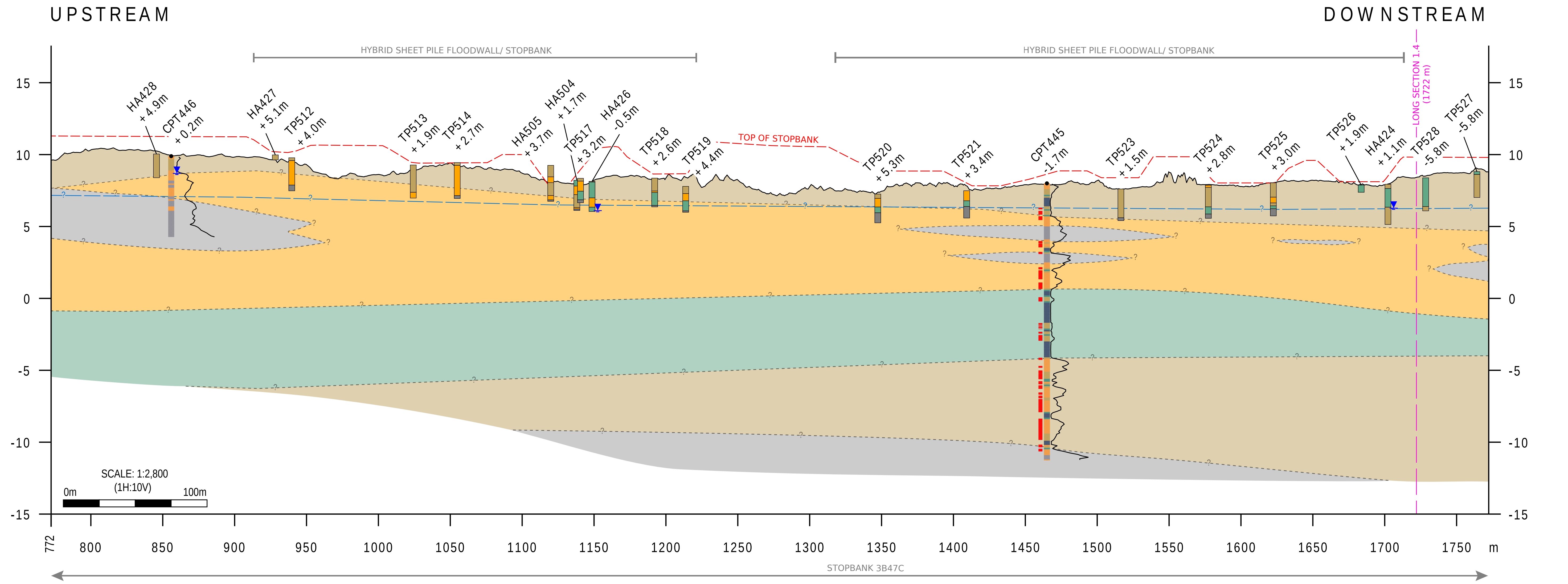
CLIENT	HAWKES BAY REGIONAL COUNCIL
PROJECT	PAKOWHAI SECONDARY STOPBANK
TITLE	GEOLOGICAL LONG SECTION 1.2
SCALE (A3)	AS SHOWN
FIG No.	1.2
REV	1



LITHOLOGY

EXISTING STOPBANK FILL (SILTY SAND)

HOLOCENE RIVER DEPOSITS

CLAY DOMINATED (IC 3 TO 3.6)

SILT DOMINATED (IC 2.6 TO 3)

SILTY SAND DOMINATED (IC 2 TO 2.6)

SAND DOMINATED (IC 1.3 TO 2)

GRAVEL DOMINATED (IC < 1.3)

LONG SECTION JOIN LINE

BOREHOLE

0 SPT 'N' VALUE

CONE PENETRATION TEST

CONE RESISTANCE, qc

LIQUEFIABLE LAYERS (ULS FOS <1)

GROUNDWATER

MEASURED GROUNDWATER LEVEL (DECEMEBER 2024)

INFERRED GROUNDWATER LEVEL

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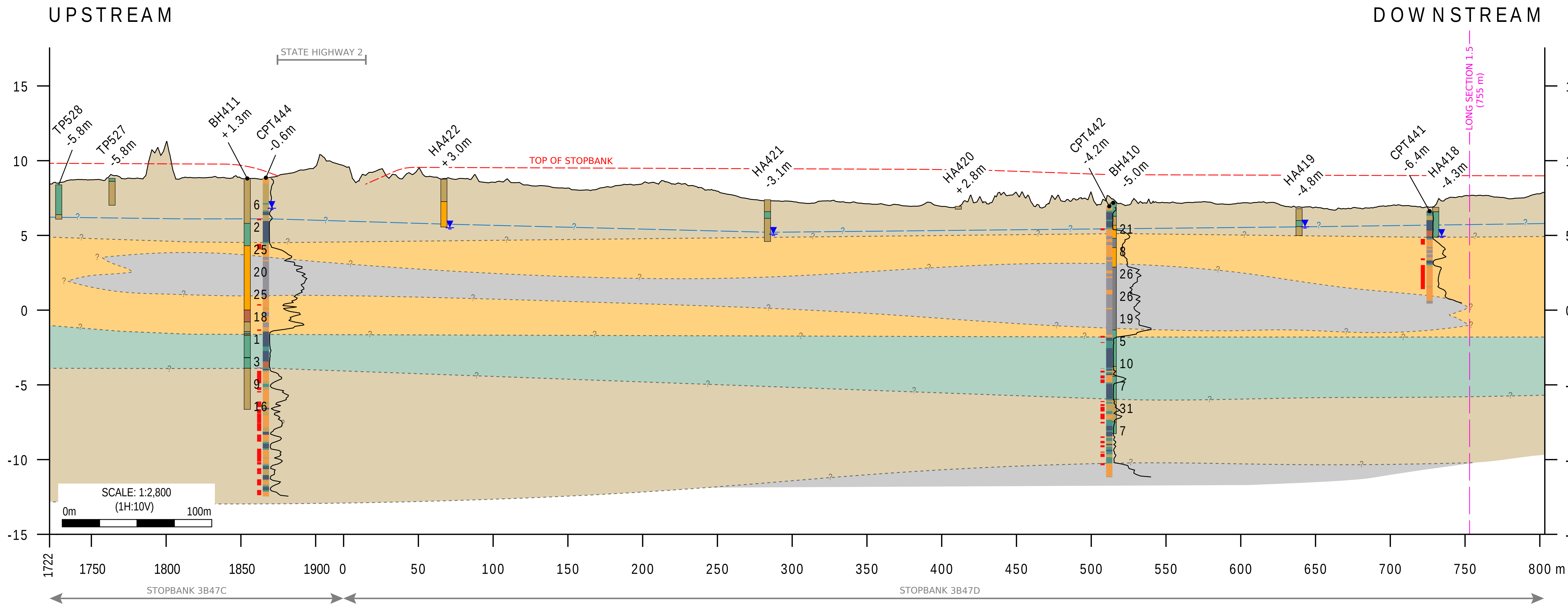
TITLE GEOLOGICAL LONG SECTION 1.3

SCALE (A3) AS SHOWN   FIG No. 1.3   REV 1



LITHOLOGY

EXISTING STOPBANK FILL (SILTY SAND)

HOLOCENE RIVER DEPOSITS

- CLAY DOMINATED (IC 3 TO 3.6)
- SILT DOMINATED (IC 2.6 TO 3)
- SILTY SAND DOMINATED (IC 2 TO 2.6)
- SAND DOMINATED (IC 1.3 TO 2)
- GRAVEL DOMINATED (IC < 1.3)
- LONG SECTION JOIN LINE

BOREHOLE

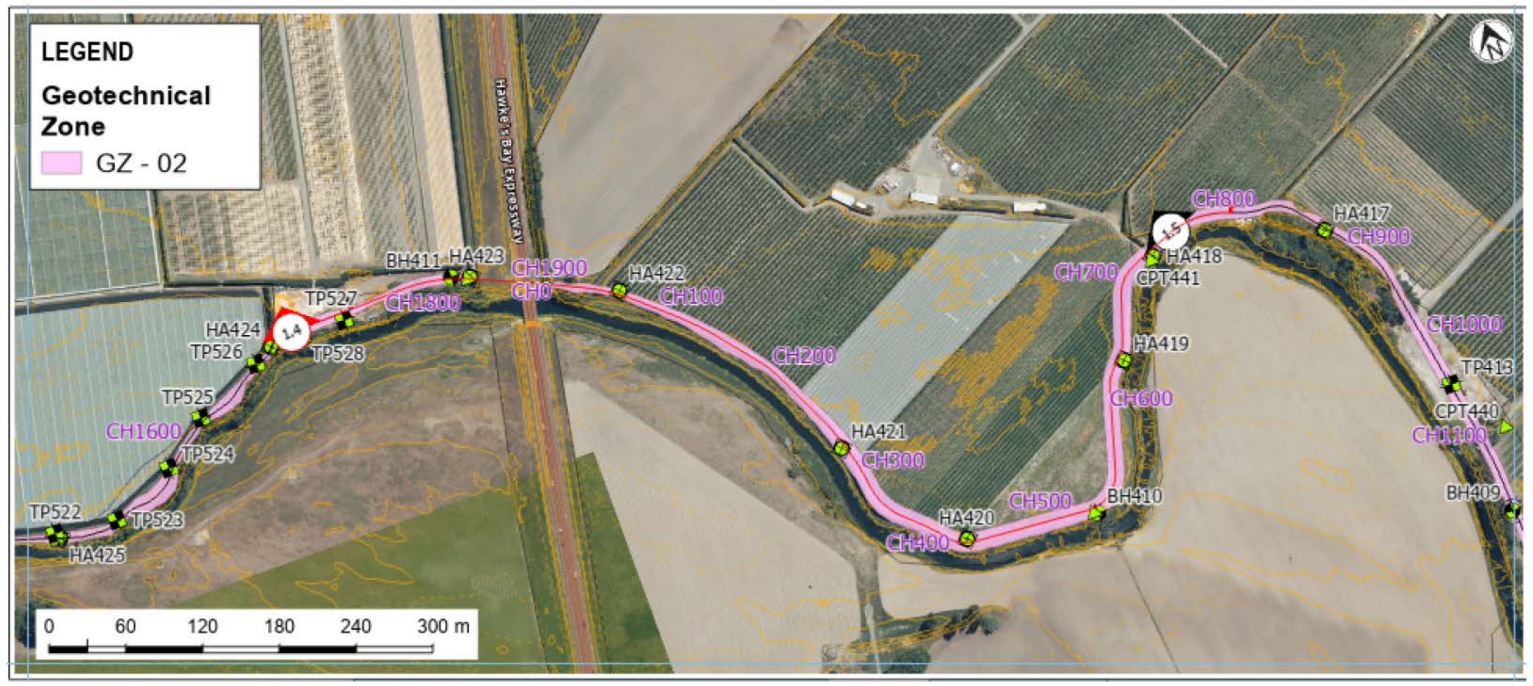
0 SPT 'N' VALUE

CONE PENETRATION TEST

- CONE RESISTANCE, qc
- LIQUEFIABLE LAYERS (ULS FOS <1)

GROUNDWATER

- MEASURED GROUNDWATER LEVEL (DECEMEBER 2024)
- INFERRED GROUNDWATER LEVEL



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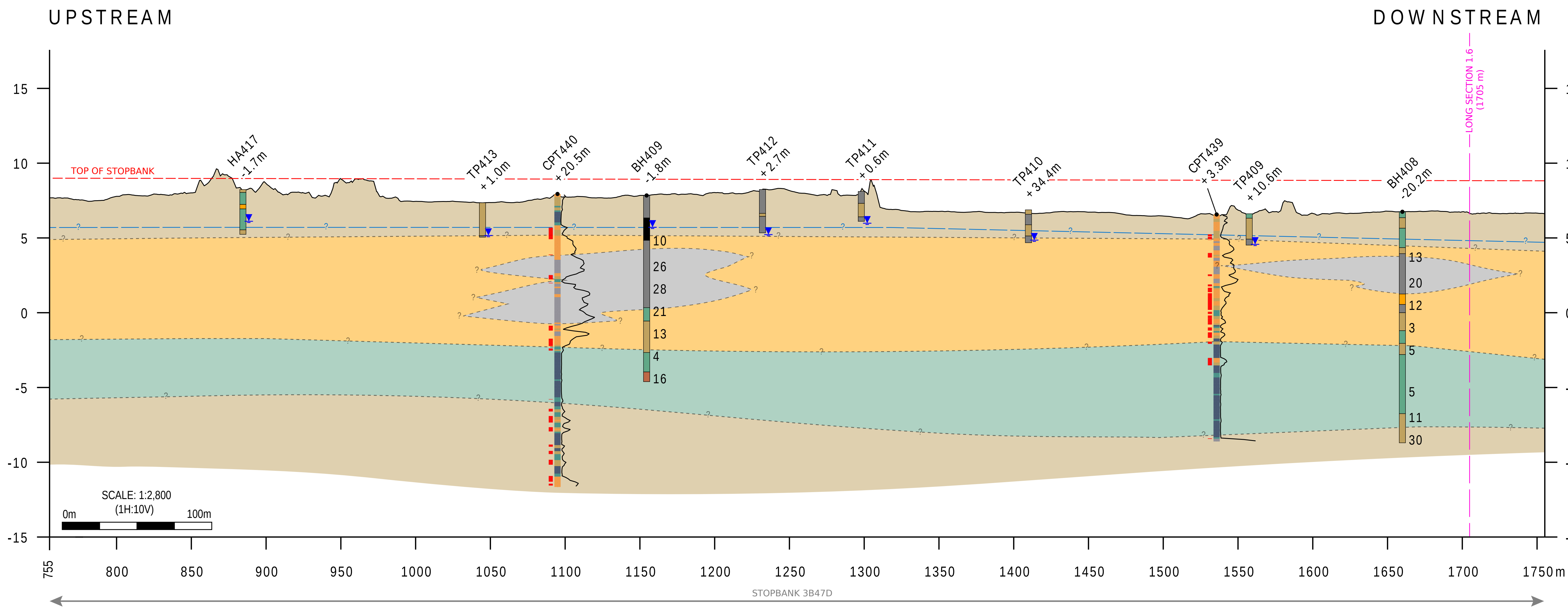
CLIENT	HAWKES BAY REGIONAL COUNCIL
PROJECT	PAKOWHAI SECONDARY STOPBANK
TITLE	GEOLOGICAL LONG SECTION 1.4
SCALE (A3)	AS SHOWN
FIG No.	1.4
REV	1



LITHOLOGY

EXISTING STOPBANK FILL (SILTY SAND)

HOLOCENE RIVER DEPOSITS

- CLAY DOMINATED (IC 3 TO 3.6)
- SILT DOMINATED (IC 2.6 TO 3)
- SILTY SAND DOMINATED (IC 2 TO 2.6)
- SAND DOMINATED (IC 1.3 TO 2)
- GRAVEL DOMINATED (IC < 1.3)
- LONG SECTION JOIN LINE

BOREHOLE

0 SPT 'N' VALUE

CONE PENETRATION TEST

- CONE RESISTANCE, qc
- LIQUEFIABLE LAYERS (ULS FOS <1)

GROUNDWATER

- MEASURED GROUNDWATER LEVEL (DECEMEBER 2024)
- INFERRED GROUNDWATER LEVEL

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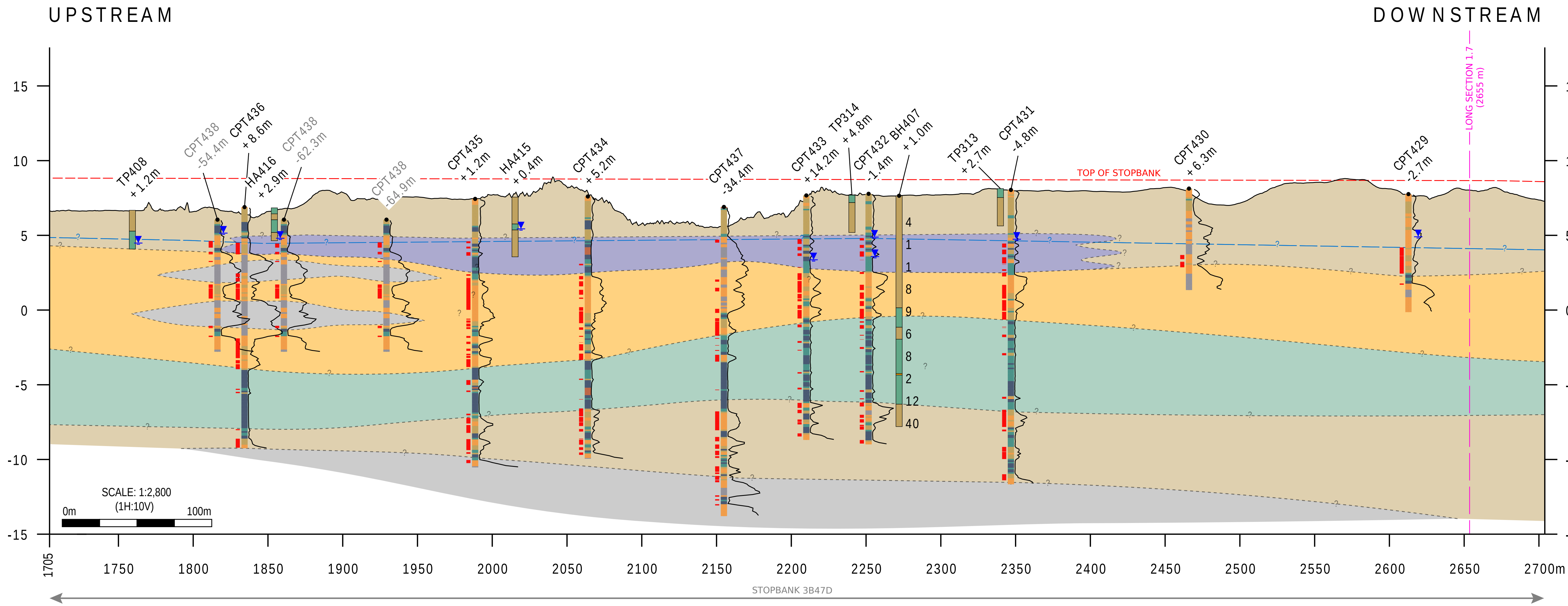
CLIENT HAWKES BAY REGIONAL COUNCIL

PROJECT PAKOWHAI SECONDARY STOPBANK

TITLE GEOLOGICAL LONG SECTION 1.5

SCALE (A3) AS SHOWN FIG No. 1.5 REV 1



LITHOLOGY

- EXISTING STOPBANK FILL (SILTY SAND)

HOLOCENE RIVER DEPOSITS

- CLAY DOMINATED (IC 3 TO 3.6)
- SILT DOMINATED (IC 2.6 TO 3)
- SILTY SAND DOMINATED (IC 2 TO 2.6)
- SAND DOMINATED (IC 1.3 TO 2)
- GRAVEL DOMINATED (IC < 1.3)
- LONG SECTION JOIN LINE

BOREHOLE

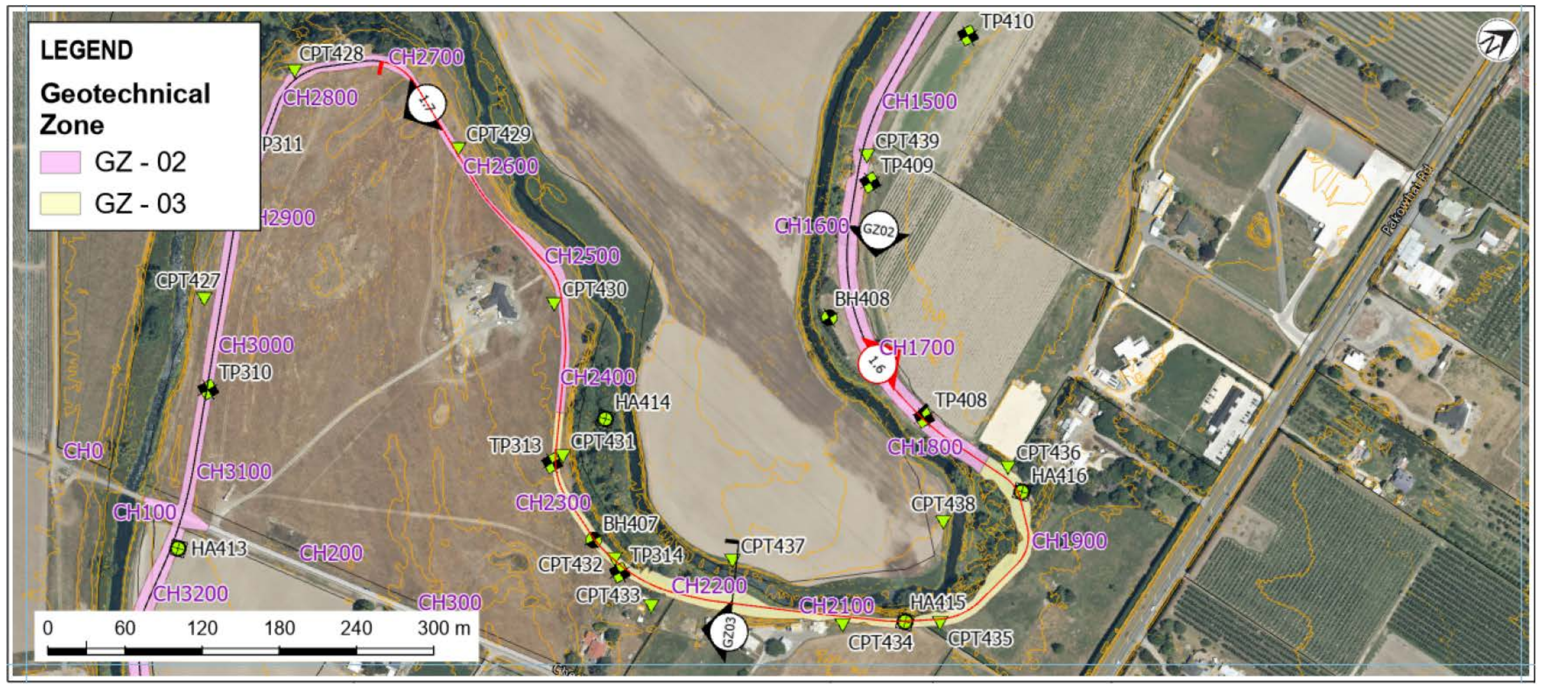
- 0 SPT 'N' VALUE

CONE PENETRATION TEST

- CONE RESISTANCE, qc
- LIQUEFIABLE LAYERS (ULS FOS <1)

GROUNDWATER

- MEASURED GROUNDWATER LEVEL (DECEMEBER 2024)
- INFERRED GROUNDWATER LEVEL



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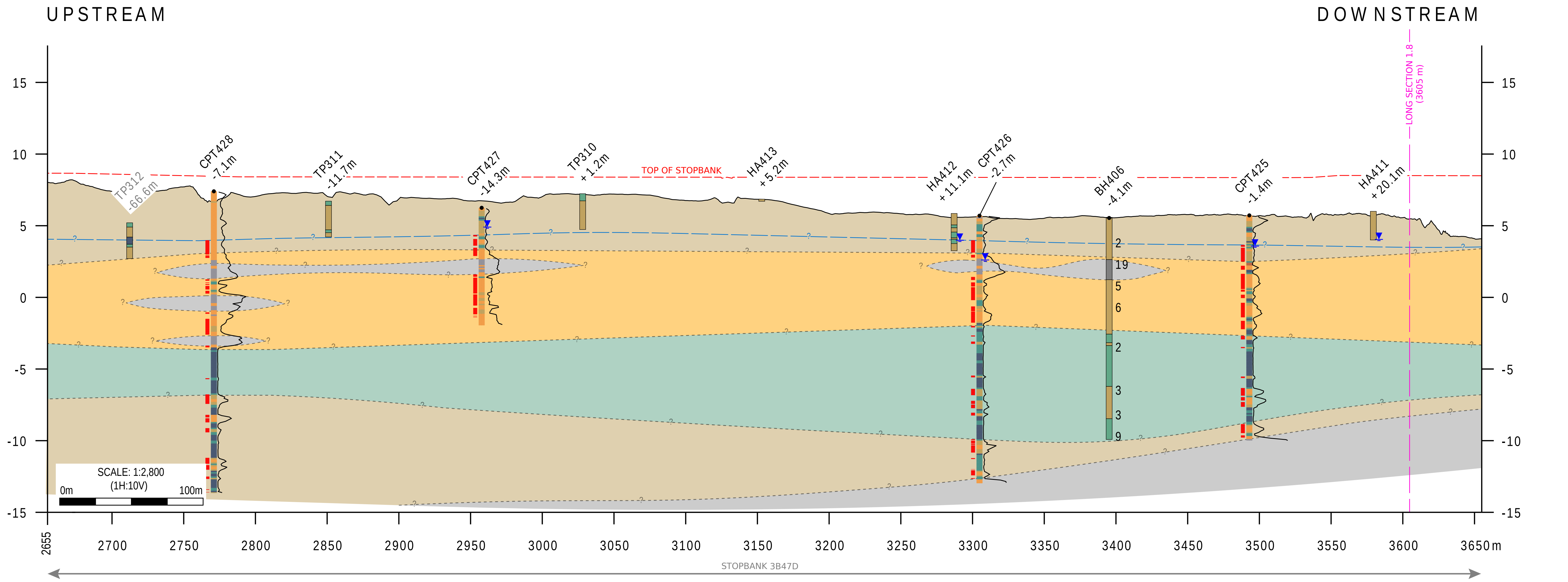
CLIENT	HAWKES BAY REGIONAL COUNCIL
PROJECT	PAKOWHAI SECONDARY STOPBANK
TITLE	GEOLOGICAL LONG SECTION 1.6
SCALE (A3)	AS SHOWN
FIG No.	1.6
REV	1



LITHOLOGY

EXISTING STOPBANK FILL (SILTY SAND)

HOLOCENE RIVER DEPOSITS

- CLAY DOMINATED (IC 3 TO 3.6)
- SILT DOMINATED (IC 2.6 TO 3)
- SILTY SAND DOMINATED (IC 2 TO 2.6)
- SAND DOMINATED (IC 1.3 TO 2)
- GRAVEL DOMINATED (IC < 1.3)
- LONG SECTION JOIN LINE

BOREHOLE

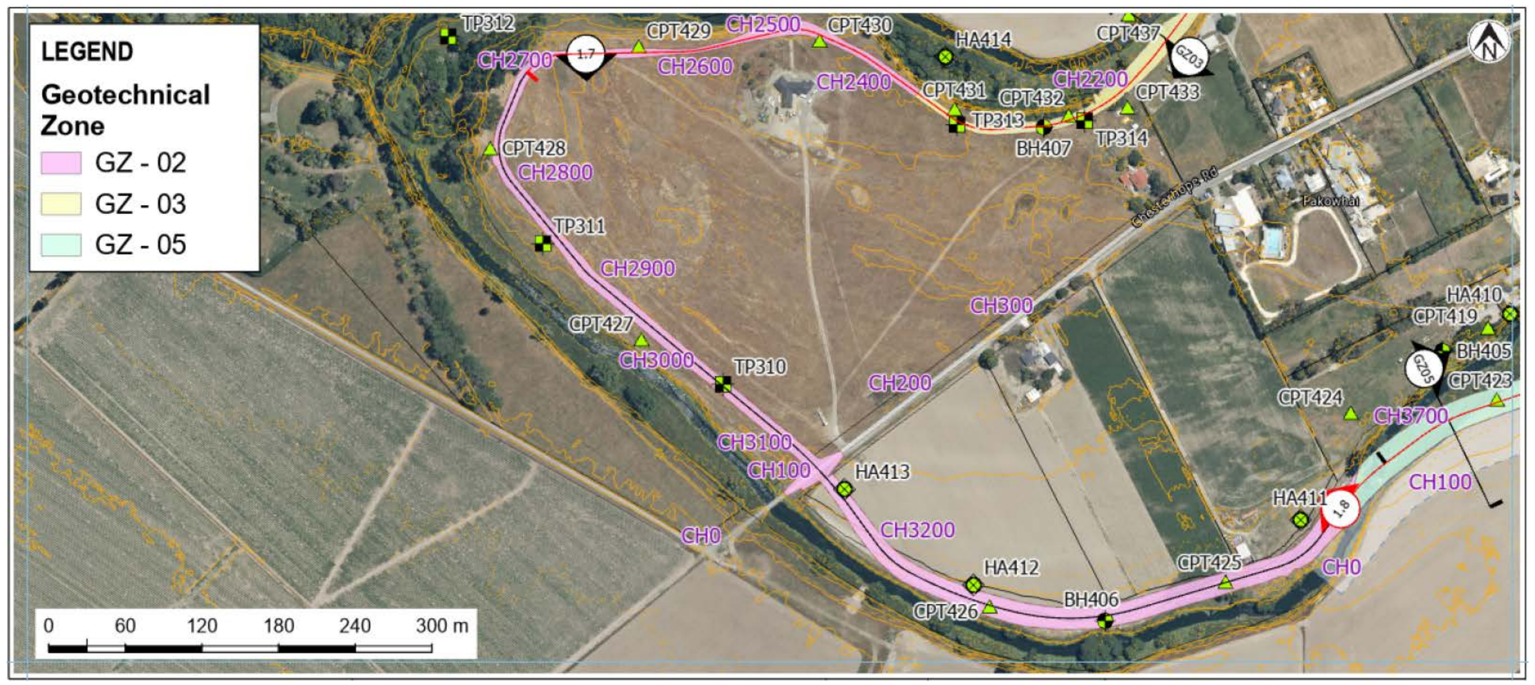
0 SPT 'N' VALUE

CONE PENETRATION TEST

- CONE RESISTANCE, qc
- LIQUEFIABLE LAYERS (ULS FOS <1)

GROUNDWATER

- MEASURED GROUNDWATER LEVEL (DECEMEBER 2024)
- INFERRED GROUNDWATER LEVEL



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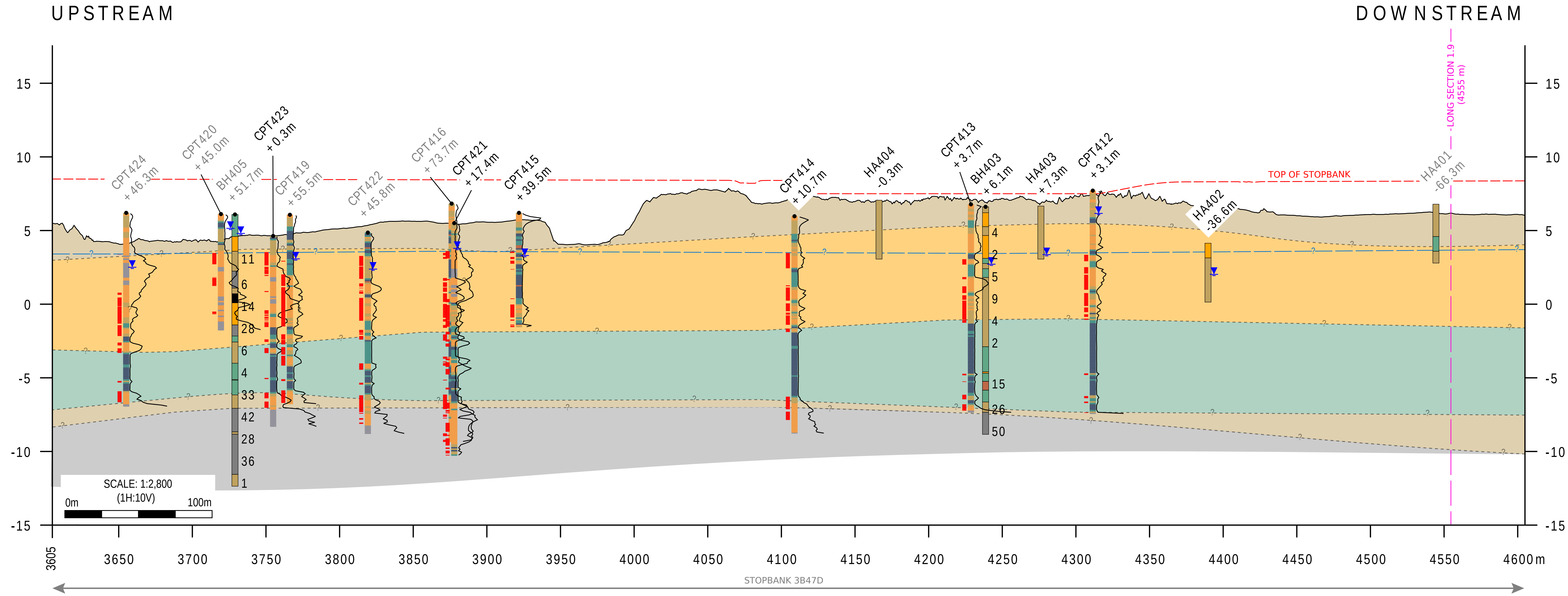
CLIENT	HAWKES BAY REGIONAL COUNCIL
PROJECT	PAKOWHAI SECONDARY STOPBANK
TITLE	GEOLOGICAL LONG SECTION 1.7
SCALE (A3)	AS SHOWN
FIG No.	1.7
REV	1



LITHOLOGY

EXISTING STOPBANK FILL (SILTY SAND)

HOLOCENE RIVER DEPOSITS

- CLAY DOMINATED (IC 3 TO 3.6)
- SILT DOMINATED (IC 2.6 TO 3)
- SILTY SAND DOMINATED (IC 2 TO 2.6)
- SAND DOMINATED (IC 1.3 TO 2)
- GRAVEL DOMINATED (IC < 1.3)
- LONG SECTION JOIN LINE

BOREHOLE

0 SPT 'N' VALUE

CONE PENETRATION TEST

- CONE RESISTANCE, qc
- LIQUEFIABLE LAYERS (ULS FOS <1)

GROUNDWATER

- MEASURED GROUNDWATER LEVEL (DECEMEBER 2024)
- INFERRED GROUNDWATER LEVEL

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2. INVESTIGATIONS OVER 40M OFFSET FROM THE SECTION ALIGNMENT HAVE BEEN COLOUR GREY
3. ELEVATION LEVELS ARE WITHIN NZVD16 VERTICAL DATUM
4. WE HAVE GROUPED THE UNITS BASED ON THE DOMINANT GEOLOGICAL UNIT AND WHAT IS LIKELY TO LEAD TO MORE CONSERVATIVE CONDITIONS IN OUR MODELLING

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PROJECT No.	1017353.2403	
DESIGNED	BRTA	Aug.25
DRAWN	BRTA	Aug.25
CHECKED	DAMI	Sep.25
APPROVED		DATE

CLIENT	HAWKES BAY REGIONAL COUNCIL
PROJECT	PAKOWHAI SECONDARY STOPBANK
TITLE	GEOLOGICAL LONG SECTION 1.8
SCALE (A3)	AS SHOWN
FIG No.	1.8
REV	1