APPENDIX 15: GEOTECHNICAL REPORT

Te Karaka Stopbank Upgrade

Geotechnical Interpretive Report

Gisborne District Council

31 March 2026

→ **The Power of Commitment**

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<tr><td>S3</td><td>Rev0</td><td>Mel Kleyburg
Paula Ramirez</td><td>Sam Harris</td><td></td><td>Darren Maxwell</td><td></td><td>20/3/2026</td></tr>
<tr><td>S4</td><td>Rev1</td><td>Mel Kleyburg
Paula Ramirez</td><td>Sam Harris</td><td></td><td>Darren Maxwell</td><td></td><td>31/3/2026</td></tr>
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1. Introduction

1.1 General

GHD Limited (GHD) have been engaged by Gisborne District Council (GDC) to undertake a Geotechnical Assessment to inform the design of proposed stopbank upgrades at Te Karaka, Gisborne. These proposed stopbank upgrades are part of the Te Karaka Flood Protection Scheme which has an overall aim to upgrade and improve the Te Karaka township's flood defences[1].

A Geotechnical Factual Report (GFR) was previously prepared by GHD (dated 9th February 2026). The purpose of this Geotechnical Interpretive Report (GIR) is to present the results of the Geotechnical Assessment. We understand that this GIR may be used to support Resource Consent. This GIR should be read in conjunction with the GFR.

1.2 Proposed works

The proposed Te Karaka Stopbank Upgrade comprises approximately 4.32 km of new and existing stopbank upgrade adjacent to the Waipaoa River and aims to provide a similar level of service to the (we understand) recently completed downstream Waipaoa Stopbank Upgrades.

The proposed stopbank alignment has been divided into the Eastern Alignment (Chainage 0 to 1301) and Western Alignment (includes Chainage 1744 to 4360). The approximate height of the proposed stopbank is approximately 2.0 to 5.0 m above existing ground level. Resource Consent level Drawings (not for Construction) of the proposed alignment are presented in Appendix A. The approximate location of the existing and new stopbank is presented in Figure 1.

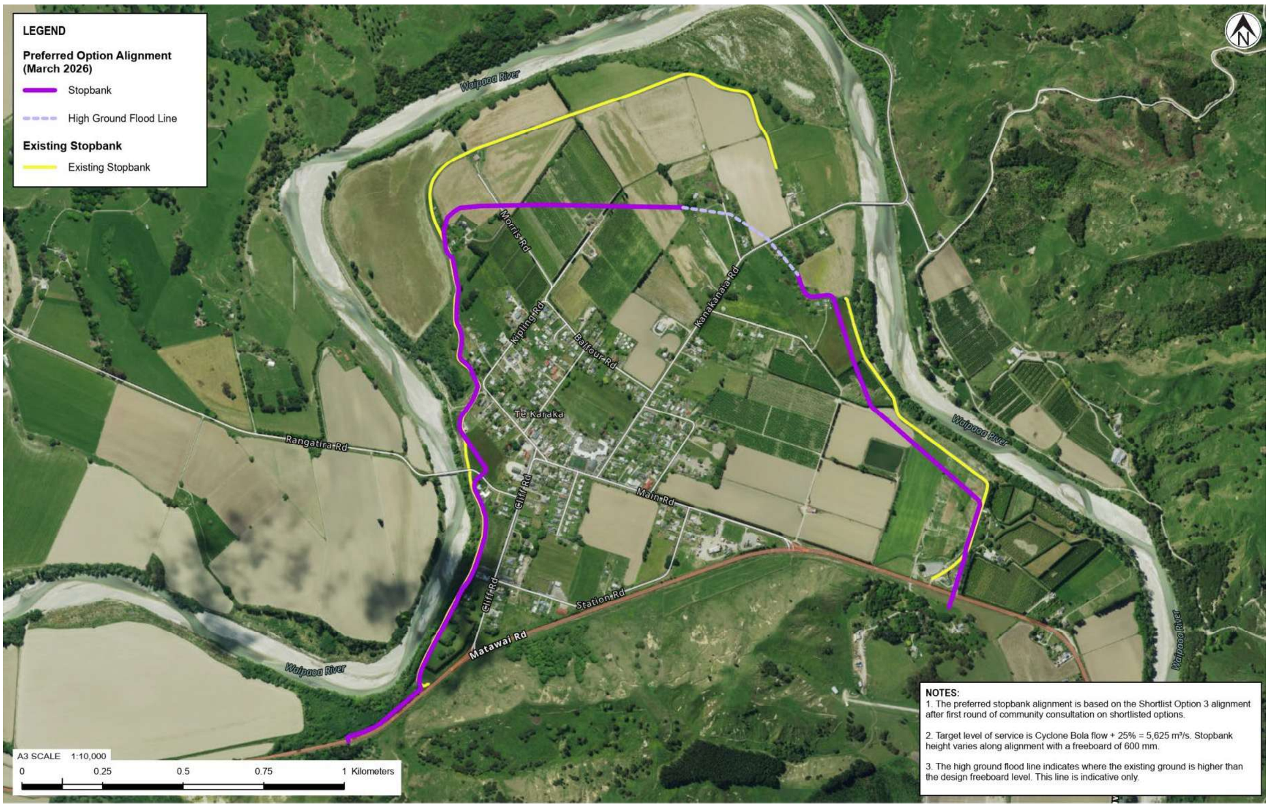

Figure 1 ***Te Karaka Stopbank site location (extract from drawing provided by GDC).***

[1] **https://www.gdc.govt.nz/our-recovery/flood-resilience/te-karaka-flood-resilience-improvements**

1.3 Scope of services

This report is intended to present the results of a geotechnical assessment and provide geotechnical recommendations and design considerations to inform design of the proposed works.

The scope of this report includes:

– Review of existing stopbank design drawings, site geomorphology and site investigation results to identify locations where geotechnical risks (namely settlement/soft ground conditions and slope stability) may need to be considered in the detailed design of the stopbank.
– Develop a two-dimensional ground model and geotechnical parameters at three locations based on the site investigation results and survey data provided by GDC.
– Carry out initial slope stability analysis using SLOPE/W two-dimensional limit equilibrium software and assumed flood level scenarios.
– Identify risks, potential mitigation strategies and opportunities associated with the existing stopbank alignment/design. It is intended this GIR will be used to support a resource consent application.

1.4 Site description

The site is located in Te Karaka, inland approximately 30 km north-west of Gisborne. Te Karaka is a small rural township located on a floodplain and surrounded by the Waipaoa River. Te Karaka is bounded by State Highway 2 (SH2) / Matawai Road to the south.

Te Karaka and its surrounds is predominantly used for agricultural purposes (pastoral farming) and forestry. The terrain is relatively flat, with elevations ranging from 38 metres reduced level (m RL) to 42 m RL. At the property 81 Kanakanaia Road there is a small hill with an elevation increase 40 m RL to 53 m RL.

Other notable features include a 70 m wide by 100 m long effluent pond located on the eastern side, which is located approximately 70 m from the existing stopbank at Chainage 640. An old train tunnel is located by State Highway 2, with the entrance approximately 200 m south of start of the western alignment stopbank. The entrance appears to remain open, although has not specifically been inspected to confirm if any plug / blockage exists.

1.5 Existing services

A review of BeforeUdig and the GDC GIS website[2] indicates the presence of existing wastewater, water supply, stormwater, and gas services within and adjacent to the Te Karaka township and along sections of the proposed stopbank alignment. Refer to Figure 2which provides an indicative location of known utility services.

A desktop comparison of the proposed stopbank alignment (per Figure 1) and the existing services, indicates that several sections of the alignment traverse or run adjacent to existing buried services.

Localised service interaction areas were identified and have been circled in Figure 2 (green circles). Some of the potential interaction areas have been listed below:

- Wastewater pipelines near the Waipaoa River
- Water supply and stormwater services along road corridors
- Main gas line running along Matawai Road and services adjacent to residential properties
- Overhead power lines are observed in the proximity of alignment.

[2] **https://maps.gdc.govt.nz/H5V2_12/** (Reviewed 10/03/2026)

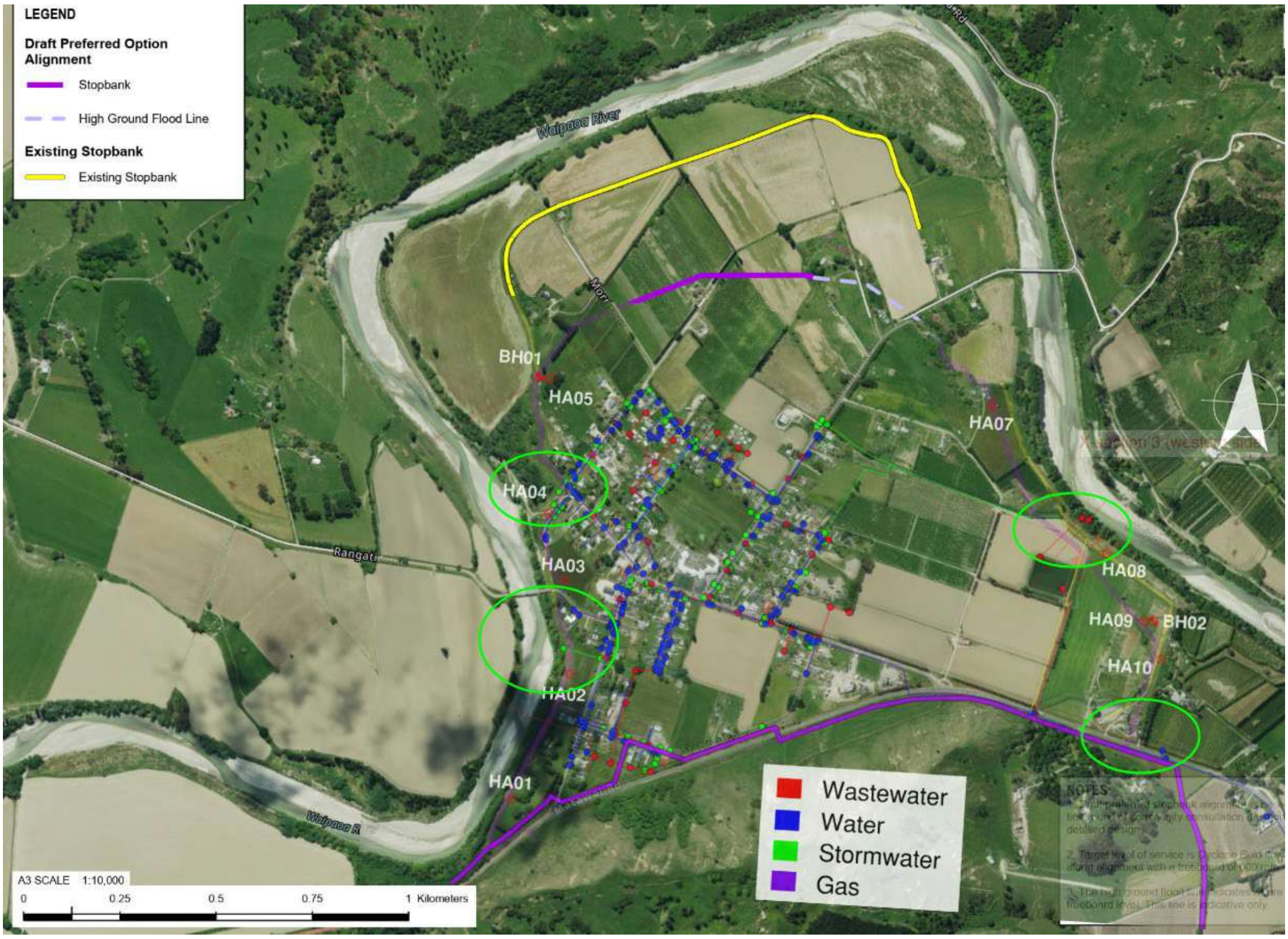

Figure 2 *Te Karaka indicative services plan. Green circles indicate potential service clashes with stopbank.*

1.6 Published Geology

The published 1:250,000 geological map of the Raukumara area (Mazengarb and Speden, 2000) shows the surficial geology of the sites to comprise of Holocene river deposits (Q1a) and a pocket of Late Pleistocene river deposits (Q3a) of the Pakihi Supergroup. An extract of this map is presented in Figure 3 and shows the geology of the sites and surrounding area, with the units present described in Table 1 (Mazengarb and Speden, 2000).

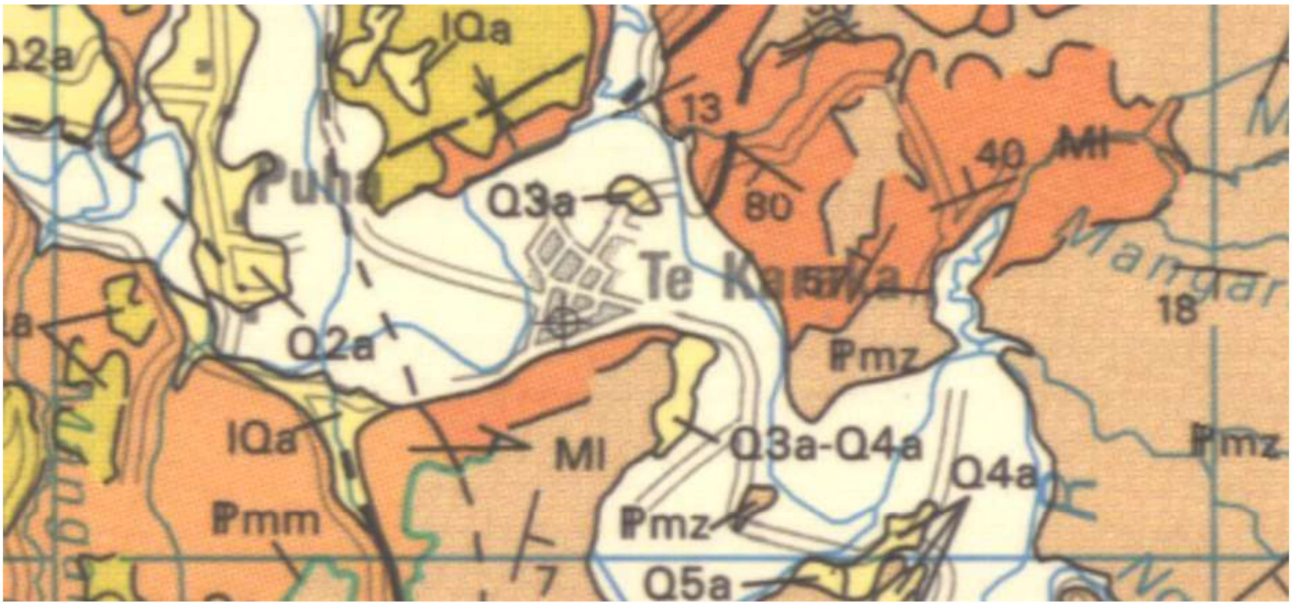

Figure 3 **Extract of the 1:250,000 geological map of the Raukumara area.**

Table 1 **Extract of the 1:250,000 geological map legend of the Raukumara area.**

Map symbol	Geological unit	Description
Q1a	Holocene river deposits (Q1a)	Poorly to moderately sorted gravel with minor sand and mud, overlain by tephra
Q3a	Pleistocene river deposits (Q3a)	Poorly to moderately sorted gravel with minor sand and mud, overlain by tephra and loess

2. Geotechnical data

2.1 NZGD review

A review of existing ground investigations from the New Zealand Geotechnical Database (NZGD) was undertaken for a radius approximately 1 km from Te Karaka and available data comprised hand augered boreholes. These investigations were conducted by Engineering Design Consultants in 2023 and generally extend to depths between 1.5 m below ground level (m BGL) and 2.5 m BGL. The logs indicate near-surface fill overlying predominantly fine-grained Holocene alluvial soils, comprising silts and clayey silts of low to high plasticity, typically described as firm to very stiff. Interbedded pumiceous sands and silty sands were also encountered locally, often at shallow depths and commonly in a saturated condition. Groundwater was encountered in several hand augers at depths generally between about 0.6 m BGL and 1.3 m BGL.

The subsoils encountered within these investigations was noted as consistent with the published geology, however it was noted these reported groundwater levels were higher than those noted in Sections 2.3 and 2.4. We presume it was due to the storm events related to Cyclone Gabrielle as the hand augers were carried out February 2023.

2.2 GDC existing geotechnical data

GDC provided ground investigation data for the proposed stopbank fill material. The information provided comprises three hand augered drillholes undertaken by WSP in May 2025, approximately 500 m northeast of the central section of the proposed stopbank alignment. The location of HA01, HA02, and HA03 is shown in Figure 4 and logs are included within Appendix B.

Figure 4 *Location of WSP investigations – proposed fill material*

The hand auger logs indicated near-surface silty soils with minor sand and clay, underlain by predominantly clayey silt and silty clay to the termination depth of approximately 2 m BGL. The soils were generally described as firm, becoming stiff with depth, and exhibit moderate to high plasticity. Minor sand content was present throughout, locally with coarse sand grains evident. Groundwater was not encountered within the investigations. The soil description presented on the logs were consistent with fine-grained alluvial soils typical of the Te Karaka area.

2.3 GDC monitoring well

The GDC groundwater monitoring website[3] was reviewed to identify groundwater information relevant to the Te Karaka area. Two groundwater monitoring wells (GP0004 and GP0028) are located in the vicinity of Te Karaka township. Well GP0004 is situated to the south of Te Karaka, near the Waipaoa River, and Well GP0028 is located to the east of Te Karaka township. Figure 5 and Table 2 shows the location and well details respectively.

GHD requested the logs for both wells, however these were unavailable at the time of GIR preparation.

Table 2 *GDC Well details.*

Well ID	Approximate location relative to Te Karaka	Well depth (m)	Casing diameter (mm)	Screen interval (m BGL)	Reported groundwater level (m BGL)
GP0004	South of Te Karaka township, near the Waipaoa River	~9.1	100	Not reported	Not reported
GP0028	East of Te Karaka township	~13.6	100	~12.1 to 13.6	~8.5

[3] **https://www.lawa.org.nz/explore-data/groundwater-quality#/tb-region** (Reviewed: 10/03/2026)

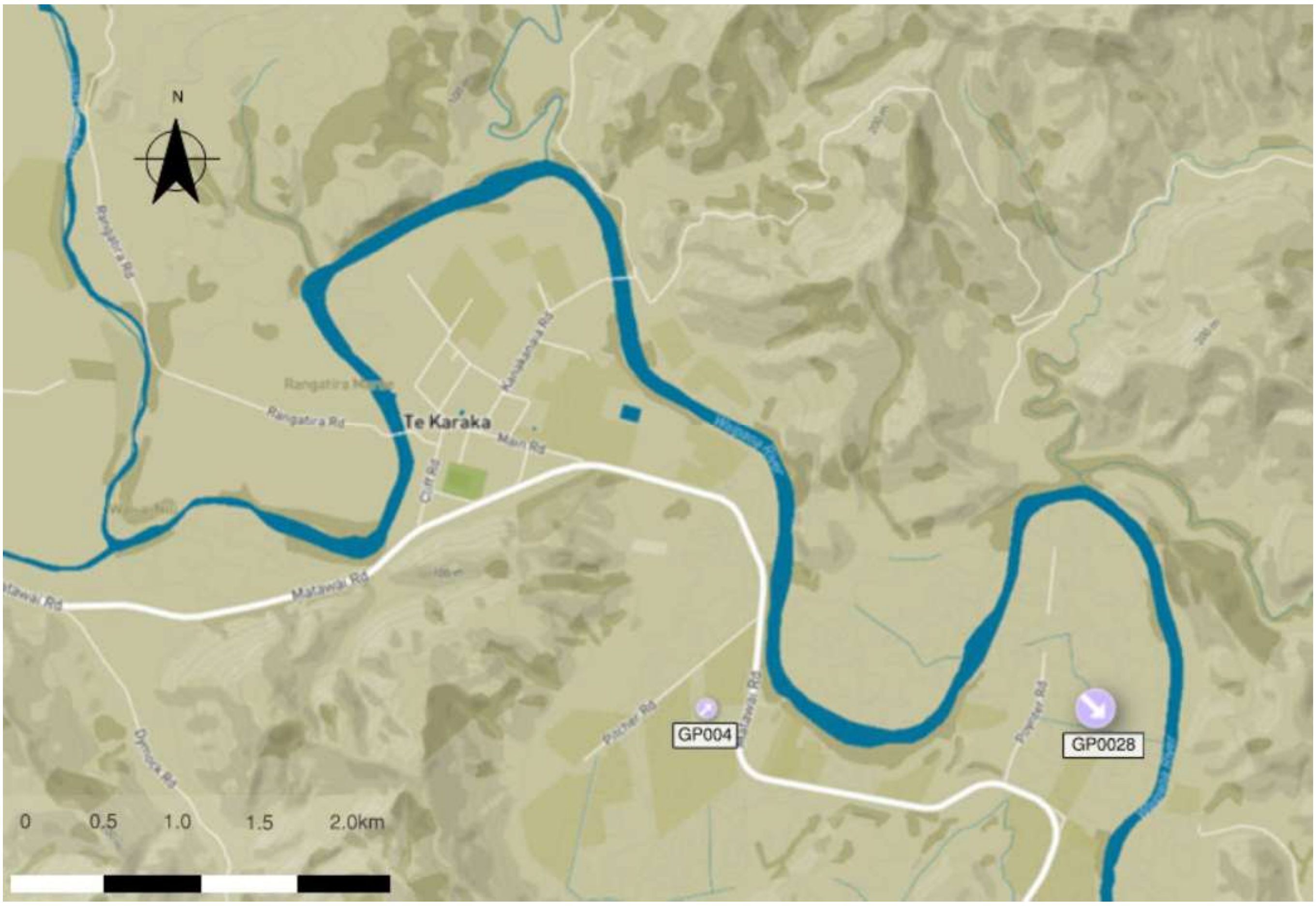

Figure 5 GDC Well locations.

2.4 GHD ground investigations (2025)

GHD conducted geotechnical investigations between the 17th and 19th of December 2025. The investigations were undertaken along the proposed (at the time of preparation) Te Karaka Stopbank alignment to characterise subsurface conditions and inform design. The investigation comprised:

– Two machine boreholes were drilled to depths of approximately 10.95 m BGL, and piezometers were installed in both boreholes
– Nine hand augers which were advanced to depths of between 0.2 m BGL and 2.2 m BGL at various locations
– Standard Penetration Tests (SPT) were carried out in the boreholes at 1.5 m depth intervals, and shear vane testing was undertaken in cohesive soils where practicable.

A location map of the geotechnical investigations, borehole logs, core photography and calibration certificates for the relevant testing equipment contained within the GFR.

3. Engineering geological assessment

3.1 Ground conditions

The results of the ground investigation indicate the ground conditions to be topsoil/fill up to 0.3 m BGL followed by interbedded silt and sand, with low plasticity cohesive soils predominating in the upper profile. These materials are typically stiff to very stiff to the base of the hand augers up to 2 m bgl. In the deeper boreholes (BH CH2560 and BH CH380), sandy silts and silty sands extend to depths of approximately 7 m BGL to 9 m BGL, underlain locally by dense sand or gravel layers. A dense gravel layer was encountered at 10.5 m BGL in BH CH2560.

A summary of the anticipated ground conditions across the study area indicates that the proposed alignment will pass through Holocene River Deposits. This formation has been separated into two general engineering geological sub-units, with different engineering characteristics as summarised in Table 3.

Table 3 Summary of ground conditions

Geological Unit	Description	Observations	Typical thickness	Field test results (indicative)
Existing Stopbank Fill / Topsoil	[1] Silt with some sand	- Near-surface fill or reworked soils comprising silt with some sand. - Stiff to very stiff.	~0.2–0.5 m (locally up to ~1.0 m)	Hand auger refusal common at shallow depth. Shear vane values up to 200 kPa.
Holocene deposits	[2] Silty sand / sandy silt – Undrained	- Interbedded sandy silt and silty sand. - Low plasticity cohesive soils with sand partings. - Firm to stiff.	~6–9 m	SPT N: typically 2–7 Shear vane: ~40–100 kPa
	[3] Fine to coarse gravel	Sand and gravel; locally dense; grey; encountered at depth and not laterally continuous.	≥2 m (extends beyond investigation depth locally)	SPT N: up to ~30–33 (dense). No vane testing undertaken.
Notes: SV = field vane shear strength. SPT = Standard Penetration Test Engineering geological units are interpreted from borehole and hand auger logs and represent generalised ground conditions across the site.				

3.2 Groundwater conditions

During the 2025 GHD ground investigations, groundwater monitoring was carried out in boreholes BH CH2560 and BH CH380. The observations indicate a fluctuating groundwater table between approximately 5.8 m BGL and 7.8 m BGL. The groundwater levels encountered during the GHD geotechnical investigations are consistent with the recorded groundwater levels at the GDC monitoring well GP0028.

The 2023 EDC investigations encountered comparatively shallower groundwater levels which indicates the possibility of localised, perched groundwater along the proposed stopbank alignment.

For preliminary design purposes, the groundwater level of approximately 7.5 m BGL was adopted. Groundwater levels will be further refined during subsequent stages of design as seepage and flood modelling is developed.

3.3 Seismicity

The nearest active fault to the site recorded on the GNS Active Faults Database is the Otoko-Totangi Fault, which is located approximately 6.5 km southwest of Te Karaka. The fault is classified as active, with normal faulting as the dominant movement sense and down-throw to the south. The geological history indicates possible Pliocene and Quaternary activity; however, the total slip and maximum likely earthquake magnitude are not defined. Information on recurrence interval is not available for this fault (GNS, 2024).

3.4 Geomorphology

Te Karaka is situated on a low-lying alluvial floodplain adjacent to the Waipaoa River. The local geomorphology is dominated by broad, gently undulating floodplain surfaces formed by historical overbank flooding and lateral channel migration.

The floodplain comprises recent Holocene alluvial sediments, with subtle variations in ground level associated with former channel margins, floodplain backswamps, and abandoned channels. River terraces and higher ground are present locally beyond the immediate floodplain margins, particularly toward the eastern and southern extents of the township, where land rises gently away from the river corridor.

The proposed stopbank alignment generally follows the margin of the active floodplain. The geomorphological setting reflects ongoing fluvial processes, including sediment deposition and localised erosion, which are key considerations for stopbank design, foundation performance, and long-term flood protection resilience.

4. Geotechnical assessment

4.1 Assumptions and considerations

The following assumptions have been made as part of the preliminary assessment:

– The proposed stopbank comprises a 4 m wide crest with 1V:2H embankment slopes
– The stopbank will comprise a homogeneous fill material was modelled
– The provided draft plans indicate that a freeboard of 600 mm has been adopted along the alignment
– Cross-sections have been developed based on information provided by third parties, elevations and crest levels have not been cross checked
– Groundwater levels have been established based on measurements obtained from groundwater monitoring undertaken during the site investigations.
– We note that the chainages discussed/referred to in this report, and the GFR, may refer to outdated/previous chainages and plans (which are presented in Appendix A). We understand that since the issuing of this report, the proposed stopbank alignment has been altered slightly. The updated plans are presented in Appendix D. We do not consider this proposed change in alignment will have a significant impact to the geotechnical assessment presented herein.

4.2 Ground model

A ground model was developed for the three representative cross-sections along the existing and proposed stopbank alignments. These sections are intended to characterise the subsurface conditions across the alignment and to inform the assessment of the existing and proposed stopbank configurations.

The ground models were based on the findings of the GHD geotechnical investigations undertaken in the vicinity of the assessed sections, with geological units extrapolated between investigation locations where required. The cross section locations are shown in Appendix A, and the corresponding ground models are in Appendix C.

The following uncertainties apply with respect to the ground conditions described herein:

– The vertical extent of the ground model is limited to the depth of the geotechnical investigations undertaken by GHD in December 2025.
– Groundwater conditions are based on the design groundwater level previously defined in Section 3.2.
– Investigation locations are offset from the section alignments (per Section 4.3) and geological unit boundaries were extrapolated between boreholes and test locations.
– Natural spatial variability in subsurface conditions across the alignment may not be fully captured by the available investigation data.

4.3 Selected cross sections

The cross-section profiles were derived from LiDAR and GDC data, with both the existing stopbank and the proposed stopbank geometry incorporated into the models. The assessed sections are summarised as follows:

– Section 1 (approximate CH3800) is located on the western side of the alignment and runs generally west to east. This section incorporates investigations HA CH3790 and HA CH3390 (refer to the GFR for investigation locations).

– Section 2 (approximate CH2560) is located within the central portion of the alignment and trends from west to northeast. This section is informed by BH CH2560 and HA CH620. This section was chosen due to its proximity to available geotechnical data, and it being representative of the central portion of the proposed alignment.
– Section 3 (approximate CH680) is located on the eastern side of the alignment and trends from northeast to southwest. This section incorporates BH CH380 and HA CH1130. This section was chosen due to its proximity to an existing effluent pond.

4.4 Proposed fill material

As discussed Section 2.2, the proposed stopbank fill material is to be sourced locally from land adjacent to the Waipaoa River. Based on the description presented on the hand auger logs, the material was described as a cohesive silty clay with minor sand content. Laboratory and field descriptions indicate the material is moderately to highly plastic, with no evidence of organic soils. We consider that the proposed fill material may be suitable to be used as earthfill within stopbank embankments, given the cohesive nature and plasticity of the material is likely to provide sufficient shear strength and low permeability. However additional analysis (seepage analysis) and material testing may be required to confirm compaction requirements.

4.5 Seismic considerations

Seismic considerations for the proposed stopbank were assessed in accordance with NZS 1170, with reference to the 2021 Bay of Plenty Regional Council (BOPRC) Stopbank Design and Construction Guidelines, which provide guidance on seismic performance requirements for flood protection structures. Based on the potential consequences of failure and the level of protection provided to rural land and residential areas within Te Karaka township, the stopbank is considered an Importance Level 2 (IL2) structure. Consistent with NZS 1170.5 and the BOPRC guidelines, the seismic design actions adopted for the preliminary assessment correspond to the serviceability limit state (SLS) and ultimate limit state (ULS) earthquake events applicable to IL2.

The seismic input parameters adopted for the assessment are summarised in Table 4.

Table 4 Input parameters for determination of peak ground acceleration

Seismic case	Input parameter	Parameter value	Comment
General	Importance Level (IL)	2	NZS 1170.0
	ULS return period (years) PGA	1/500 0.65g	NZS 1170.0 Table 3.3 MBIE 2021 Table A1
	SLS return period (years) PGA	1/25 0.12g	NZS 1170.0 Table 3.3 MBIE 2021 Table A1
	Effective magnitude earthquake	6.3 (SLS) 7.5 (ULS)	MBIE 2021 Table A1

4.6 Geotechnical design parameters

The adopted geotechnical material parameters for the site are presented in Table 5, for the units that were defined in the ground model (Appendix C and Section 5.4). These parameters are based on the interpretation of the results of the investigations carried out onsite, empirical relationships and local experience. These parameters may change as design progresses and further assessment warrants.

Table 5 Geotechnical design parameters

Geological subunit	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Undrained shear strength Su (kPa)
[0] Engineered Fill – silty CLAY with minor sand - Proposed stop bank fill (assumed 95% maximum dry density)	18	2	30	100
[1] SILT with some sand [FILL]	18	0	32	N/A
[2] SILT / Sandy SILT [Holocene Deposits]	16	2	28	50
[3] Fine to coarse GRAVEL [Holocene Deposits]	20	0	40	N/A

4.7 Stability analysis

4.7.1 Methodology and criteria

The slope stability analyses was undertaken using GeoStudio 2025.1.1 SLOPE/W software for the cross-sections described in Section 4.3. The following assumptions and methodology was applied:

- The Morgenstern–Price (1967) limit equilibrium method was used to calculate factors of safety (FoS).
- An entry–exit slip surface search was adopted.
- The stopbank geometry was modelled in accordance with the provided draft alignment and the assumptions outlined in this report.
- No surcharge loading was applied at the crest of the stopbank.
- Undrained shear strength parameters were adopted for the end-of-construction and seismic loading cases, while drained shear strength parameters were applied for static and flooding conditions.
- Groundwater conditions were modelled using a piezometric surface based on the assumption that for static groundwater conditions, the piezometric surface was set as consistent with measured groundwater levels. No groundwater seepage analysis has been undertaken at this stage of the assessment.
- For seismic loading conditions, serviceability limit state (SLS) and ultimate limit state (ULS) peak ground accelerations were applied in accordance with Table 4.

4.7.2 Loading cases and assumed water levels

Load cases and target FoS criteria were based on those presented in Table A2.1 of the BOPRC guidelines. The analysed load cases included:

- Case 1: End-of-construction conditions assessed under undrained strength assumptions.
- Case 2: Rapid drawdown following peak flood levels, it was assumed that the water level at the river side will be at peak river level based on GDC data from the Waipaoa River.
- Case 3a: Long-term steady-state seepage conditions (DWSL[4]), the water level at the analysed cross-sections was assumed to be 600 mm below the current stopbank crest level.
- Case 3b: Long-term steady-state seepage conditions (HTOS[2]), where the water level is the stopbank crest.
- Case 4: Seismic loading conditions 4a and 4b for SLS and ULS respectively.

4.7.3 Summary of findings

Slope stability analyses were undertaken for the loading conditions described in Section 4.7.2. The riverside and landward stopbank slopes were analysed and stability assessment results for the three cross sections are summarised in Table 6, Table 7 and Table 8 respectively. Slope stability analysis outputs are within Appendix C.

[4] Design Water Service Level (DWSL) and Hydraulic Top of Stopbank (HTOS) are described further within the BOPRC Guidelines.

Table 6 Slope stability results – Section 1 – Western side at CH3800

Case	Description	Target FoS	River side achieved FoS	Land side achieved FoS
Case 1	End of Construction - (Undrained strength)	1.3	2.5	2.8
Case 2	Rapid drawdown	1.2	1.2	1.6
Case 3a	Long-term steady conditions -DWSL	1.5	1.6	1.6
Case 3b	Long-term steady conditions - HTOS	1.2	1.7	1.2
Case 4a	SLS (1/25 year return period)	1.0	1.7	2.8
Case 4b	ULS (1/500 year return period)	1.0	0.8	1.1
*Results in red text indicate cases where the target FoS was not met.				

Table 7 Slope stability results – Section 2 – Middle side at CH2560

Case	Description	Target FoS	River side achieved FoS	Land side achieved FoS
Case 1	End of Construction - (Undrained strength)	1.3	2.6	5.0
Case 2	Rapid drawdown	1.2	1.3	1.9
Case 3a	Long-term steady conditions -DWSL	1.5	1.6	1.7
Case 3b	Long-term steady conditions - HTOS	1.2	1.7	1.9
Case 4a	SLS (1/25)	1.0	2.0	3.0
Case 4b	ULS (1/500)	1.0	0.9	0.9
*Results in red text indicate cases where the target FoS was not met.				

Table 8 Slope stability results – Section 3 – Eastern side at CH680

Case	Description	Target FoS	River side achieved FoS	Land side achieved FoS
Case 1	End of Construction - (Undrained strength)	1.3	3.8	3.7
Case 2	Rapid drawdown	1.2	1.6	1.6
Case 3a	Long-term steady conditions -DWSL	1.5	1.7	1.5
Case 3b	Long-term steady conditions - HTOS	1.2	1.9	1.6
Case 4a	SLS (1/25)	1.0	3.0	2.8
Case 4b	ULS (1/500)	1.0	2.9	2.9
*Results in red text indicate cases where the target FoS was not met.				

4.7.4 Discussion

The stability assessment indicates that for the proposed stopbank geometry and adopted fill material parameters, the riverside slopes meet the target FoS for static, flood, and Serviceability Limit State (SLS) loading conditions. The landside slopes satisfy the static and SLS seismic criteria but do not meet the target FoS under ULS seismic cases.

The Ultimate Limit State (ULS) seismic loading case does not achieve the target factor of safety of 1.0 due to the high peak ground acceleration values; however, the ULS condition is not considered to govern the design. as it may induce some lateral displacement, the stopbank is expected to remain stable and functional under static and flood loading conditions. This means that following a large seismic event, some repairs/maintenance may be required.

This stability assessment should be updated once a seepage analysis has been undertaken (during detailed design).

4.8 Liquefaction

The GDC hazard maps (GDC, 2026) indicate that the proposed stopbank alignment is located within an area mapped as having a moderate liquefaction risk. Soils susceptible to liquefaction are typically granular, non-cohesive, and of low plasticity, with site-specific factors such as groundwater level also influencing liquefaction potential.

Based on available site information, the near-surface materials are interpreted to comprise predominantly fine-grained silts and clayey silts with moderate plasticity, which are generally not considered highly susceptible to liquefaction. Groundwater is anticipated to be encountered at depths greater than approximately 5.8 m below ground level. We consider it likely therefore that there is a non-liquefiable crust present at this site which would reduce the risk / damage induced by liquefaction in deeper layers. Notwithstanding, vertical settlement and potential lateral spreading may occur and disrupt the stopbank should liquefaction manifest in large seismic events.

In our experience, mitigation of liquefaction effects in linear infrastructure such as stop banks is cost prohibitive, and typically the risk of liquefaction is accepted, with the understanding that remedial works will likely be required.

4.9 Bearing capacity

Upon review of the geotechnical logs from the 2025 ground investigations and the ground conditions summarised in Sections 3.1 and 3.2, bearing capacity is unlikely to be a significant risk for the proposed stopbank configuration and alignment based on:

– Typical groundwater levels at depth, though noting there may be some potential of localised perched groundwater tables.
– Holocene deposits with medium density coarse grained, and firm to stiff fine grained materials as less likely to be susceptible to bearing capacity failure.

We recommend bearing capacity be checked during detailed design and following completion of seepage modelling. If needed, mitigations can include options such as drainage zones and flattening stopbank slopes.

4.10 Static settlement

We do not consider static settlement to be a significant risk given ground conditions; however we note there are some softer layers of silt present at depth (at the approximate location of 7.0 m below ground level), which may cause settlement. A detailed settlement assessment is proposed as part of the detailed design process. This will inform the likely additional fill required to achieve the target design stopbank height and inform of which nearby services/infrastructure/buildings may be at risk of settlement induced by the proposed stopbank.

Settlement can be mitigated through monitoring, or relocation of services (which may be required as part of the works for seepage reasons). Alternatively, minor realignment of the stopbank, or floodwalls, could be utilised should settlement be deemed to be a risk during the detailed design stage.

5. Considerations for detailed design

The following items should be considered for the detailed design stage. This is not an exhaustive list.

– Update the slope stability assessment to incorporate seepage modelling results, including both steady-state and transient groundwater conditions. We consider it unlikely that this assessment will significantly affect the results of the assessment presented herein.
– Undertake a piping check utilising the results of the seepage modelling.
– The interaction between the existing services and proposed works should be considered as part of the detailed design.

- Undertake settlement checks at critical locations (where the proposed new stopbank is located within close proximity to services/buildings).
- Inspect the railway tunnel and assess the risk associated with this existing tunnel. We understand that a bund is proposed around this tunnel entrance to act as a stopbank in this area.
- A safety in design assessment should be undertaken for the proposed design.
- Construction recommendations and methodology where the existing stopbank is to be raised, as particular care is required in rural areas where the embankment may comprise topsoil or other pervious materials. In such cases, stripping and benching of the existing embankment is important to reduce the risk of slippage and preferential seepage during and after construction.
- Existing topsoil will need to be stripped and stockpiled for re-use.
- Detailed design should consider maintenance and pastoral farmland requirements. Although side slopes of 1V:2H may result in adequate stopbank performance, they may create a health and safety risk (farm vehicles and maintenance vehicles), be challenging to maintain, and may be difficult for pastoral farmland use.
- At chainage 680 (on the eastern side) seepage analysis will need with the effluent pond will need to be considered for detail design.
- We note that the chainages discussed herein may be subject to change/differ from those presented on plans due to minor alignment changes. These alignments/chainages should be confirmed during the detailed design phase.

6. Limitations

This report has been prepared by GHD Limited for Gisborne District Council and may only be used and relied on by Gisborne District Council for the purpose agreed between GHD and Gisborne District Council as set out in Section 1.3 of this report.

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An understanding of the geotechnical site conditions depends on the integration of many pieces of information, some regional, some site specific, some structure specific and some experienced based. Hence this report should not be altered, amended, abbreviated, or issued in part in any way without prior written approval by GHD. GHD does not accept liability in connection with the issuing of an unapproved or modified version of this report.

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Verification of the geotechnical assumptions and/or model is an integral part of the design process - investigation, construction verification, and performance monitoring. If the revealed ground or groundwater conditions vary from those assumed or described in this report the matter should be referred back to GHD.

7. References

The following sources of information were referred to in the preparation of this report.

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