NIWE Pump Station Upgrades

Geotechnical Design Philosophy Statement

Brian Perry Civil Limited

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1. Introduction

1.1 Project overview

The NIWE Pump Station Upgrade Works form part of the Hawke's Bay Regional Council (HBRC) programme to upgrade and improve the resilience of existing stormwater pump station infrastructure across the lower Tūtaekurī River catchment in Hawkes Bay. The project includes replacement of the existing Mission Pump Station (to be renamed Awatoto Pump Station) and Pakowhai Pump Station, to improve flood protection performance, operational reliability, and compliance with current regulatory requirements.

The pump stations are located adjacent to existing stopbanks and will be founded on young alluvial and estuarine deposits. The upgrade works involve design and construction of new below-ground concrete structures, inlet and outlet structures, stopbank penetrations, buried pipelines, and associated earthworks within a geotechnically complex environment requiring careful management of ground and groundwater hazards, seismic hazards, and interfaces with existing flood protection (stopbank) assets.

1.2 Purpose of report

This Geotechnical Design Philosophy Statement (GDPS) documents the governing geotechnical design principles, performance objectives, acceptance criteria, and analytical approaches for the NIWE Pump Station Upgrade Works.

This document is intended to:

– Provide a clear and transparent basis for geotechnical design decisions;
– Support design coordination across civil, structural, mechanical and construction disciplines;
– Inform regulatory review and consenting processes; and
– Form a reference document for detailed design, peer review and construction.

1.3 Scope and limitations

1.3.1 General

This GDPS has been prepared based on available site-specific factual information and regional datasets. Subsurface conditions are inherently variable and may differ from those interpreted. Where additional information becomes available through further investigation or construction observations, the geotechnical model and design parameters may be refined in accordance with this philosophy.

This GDPS addresses the design of permanent works. Temporary works and construction methodology are the responsibility of the Contractor; however, geotechnical constraints and performance requirements relevant to temporary works are identified where they influence permanent works outcomes or adjacent assets.

This GDPS supports building and resource consenting, independent peer review, and coordination with civil, structural, mechanical, and operational design disciplines.

1.3.2 Reliance

This report has been prepared by GHD Limited (GHD) for Brian Perry Civil Limited (BPC) and may only be used and relied on by Brian Perry Civil Limited for the purpose agreed between GHD and Brian Perry Civil Limited as set out in section 1.2 of this report.

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The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

1.3.3 Accessibility of documents

If this report is required to be accessible in any other format, this can be provided by GHD upon request and at an additional cost if necessary.

2. Geotechnical design objectives

2.1 General

The Geotechnical Principal's Requirements define the overarching objectives, roles, and performance expectations governing all geotechnical works associated with the NIWE Pump Station Upgrades. These requirements apply to the investigation, design, construction, and long-term performance of geotechnical elements such as foundations, ground improvement, earthworks, retaining structures, and works in or through existing stopbanks. Formal reference to the applicable Principal's Requirements is provided in Table 1.

A fundamental objective of the Principal's Requirements is that the upgraded pump stations and associated flood protection infrastructure must not increase flood risk or geotechnical hazard to the site or surrounding assets. The works are required to remain functional during the 1% AEP flood event, maintain the integrity and level of service of existing stopbanks, and achieve defined seismic performance outcomes consistent with their Importance Level and intended design life.

In response, the geotechnical design adopts a performance-based approach consistent with NZGS/MBIE Earthquake Geotechnical Engineering Practice, recognising that controlled ground and structural deformations may occur under extreme seismic loading. Such deformations are considered acceptable provided that life safety is maintained, instability or breach of flood protection systems is avoided, essential functionality of the pump station is preserved where required, and post-event recovery and repair can be undertaken within acceptable timeframes.

The geotechnical design therefore seeks to manage geotechnical hazards – including settlement, liquefaction, lateral spreading, slope instability, and groundwater effects, such that serviceability and recoverability objectives are met. Quantitative deformation acceptance criteria, including limits on total and differential settlement, lateral displacement, and retaining wall movements, will be confirmed during detailed design in coordination with structural, hydraulic, and mechanical performance requirements.

2.2 Stopbank penetrations

Works in or through the existing stopbank are limited to localised penetrations associated with the pumping station outlet pipelines. At the location of each penetration, the stopbank will be locally removed and reinstated on a like-for-like basis, matching the geometry, profile, crest level, slopes, and zoning of the consented stopbank design prepared by others.

The reinstated stopbank section will adopt material types, placement, compaction standards, and internal zoning consistent with the consented design and compliant with the Bay of Plenty Regional Council Stopbank Design and Construction Guidelines v1.2 (2021) and CIRIA C731. No changes to the overall stopbank alignment, crest level, or level of service are proposed outside the immediate zone of influence of the penetration.

The geotechnical design will address local performance considerations arising from the penetration and reinstatement works, including seepage and internal erosion to ensure that the reinstated stopbank section does not diminish the performance or functionality of the existing flood protection system.

The global stability, flood capacity, and overall performance of the stopbank system remain governed by the consented stopbank design prepared by others. No duplication of global stopbank design checks is proposed. Any inconsistencies identified between the penetration works and the consented stopbank design will be communicated to the Principal for resolution.

3. Background information

Geotechnical design for the NIWE Pump Station Upgrade Works shall be based on existing geotechnical related information sources presented in Table 1 below.

Table 1 Background information – geotechnical basis of design

Source	Reference	Comment
Principal's Requirements	NIWE Pump Station Upgrades – Design and Build (HBRC-24-1282), Principal's Requirements, Rev 2.0. WSP, 7 November 2024.	Defines overarching project, design, and performance requirements applicable to the pump station upgrades.
	NIWE Pump Station Upgrades – Design and Build (HBRC-24-1282), Principal's Requirements – Geotechnical, Rev 2.0. WSP, 24 October 2024.	Specifies geotechnical scope, investigation, design, and performance requirements to be addressed by the works.
Geotechnical Factual Reports	Awatoto Pumpstation Geotechnical Investigation, Factual Report, Rev 1. WSP, 17 September 2024.	Presents factual geotechnical data from investigations undertaken at Awatoto in July 2024 to inform pump station development, including borehole, CPT, test pit, DCP, and SPT results.
	Pakowhai Pumpstation Geotechnical Investigation, Factual Report, Rev 1. WSP, 17 September 2024.	Presents factual geotechnical data from investigations undertaken at Pakowhai in July 2024 to inform pump station development, including borehole, CPT, test pit, DCP, and SPT results.
Geotechnical Completion Report	Geotechnical Completion Report "Gabrielle Rapid Rebuild; Awatoto", Version 4. RDCL, 28 June 2023.	Documents CPT investigations undertaken for emergency stopbank repairs following Cyclone Gabrielle; data reviewed for regional context but excluded from design ground models due to distance from site and availability of closer data.
Geotechnical interpretative reports	HBRC Awatoto Pump Station Upgrade – Geotechnical Interpretative Report, Rev1. GHD, 03 September 2025.	Geotechnical Interpretative Report prepared at Tender Design stage. Provides geotechnical interpretation using available geotechnical information, presents ground model and geotechnical assessment to inform the tender design of Awatoto Pump Station upgrade works.
	HBRC Pakowhai Pump Station Upgrade – Geotechnical Interpretative Report, Rev0. GHD, 03 March 2025.	Geotechnical Interpretative Report prepared at Tender Design stage. Provides geotechnical interpretation using available geotechnical information, presents ground model and geotechnical assessment to inform the tender design of Pakowhai Pump Station upgrade works.
Additional geotechnical investigations	Additional geotechnical investigations – design phase. GHD, February 2026.	Additional geotechnical investigations to support preliminary and detailed design at Awatoto and Pakowhai Pump Stations, comprising boreholes, CPTs, hand augers, and DCP testing. Investigation results to be included in updated Geotechnical Interpretative Reports at Preliminary and Detailed design stages.

4. Design standards and guidelines

The geotechnical design for the NIWE Pump Station Upgrades will be undertaken in accordance with the requirements of the Principal's Requirements, specifically Appendix C – Geotechnical Requirements. To achieve this the design standards, codes, and guidelines presented below in Table 2 shall be generally followed.

Table 2 Geotechnical design standards and guidelines

Design element	Design standard, code, or guideline
Site investigation	NZGS Ground Investigation Specification NZ Geotechnical Society Guidelines for Soil and Rock Description NZS 4402:1986 Methods of Testing Soils for Civil Engineering Purposes
Foundations	NZ Building Code B1/VM2 – Geotechnical design of foundations MBIE Earthquake geotechnical engineering practice Module 4: Earthquake resistant foundation design
Geotechnical earthquake engineering	New Zealand Geotechnical Society (NZGS) and Ministry of Business Innovation & Employment (MBIE) Earthquake Geotechnical Engineering Practice Module 1 to 6. NZS 1170.5:2004 Structural design actions, Part 5: Earthquake actions – New Zealand
Retaining walls	MBIE Earthquake geotechnical engineering practice Module 6: Earthquake resistant retaining wall design CIRIA C760 – Guidance on Embedded Retaining Wall Design
Slope stability (cuts and fills)	NZGS Slope Stability Guidance Units 1, 2, 3, 4, 5, 6, 7.A, 7.B, 7.C
Earthworks	NZS 4431:2022 – Engineered Fill Construction for Lightweight Structures NZGS Specification NZGS_0510 Earthworks, Version 0.2
Stopbank	Bay of Plenty Regional Council Stopbank Design and Construction Guidelines v1.2 (March 2021). New Zealand Dam Safety Guidelines, NZSOLD, 2015 CIRIA C731 (CIRIA 2013) The International Levee Handbook
Pipelines	AS/NZS 2566.1 & 2566.2 – Structural design and installation of buried flexible pipelines.

5. Design ground model

5.1 Geological setting

The Awatoto and Pakowhai Pump Station sites are located on the low-lying alluvial plains of the lower Tūtaekurī River catchment in Hawke's Bay. The sites are underlain by Holocene-age deposits of the Heretaunga Formation, comprising interbedded alluvial silts, sands, and locally gravelly materials deposited in a fluvial to estuarine environment. These natural deposits are overlain by variable thicknesses of man-made fill associated with historic stopbank construction and pump station infrastructure.

The alluvial sequence reflects a highly variable depositional environment, resulting in laterally and vertically heterogeneous soil conditions, with shallow groundwater typically present and strongly influenced by adjacent drainage channels and river levels.

5.2 Geotechnical units

5.2.1 General

Materials encountered at the pump station sites have been grouped into a series of geotechnical material units for design purposes, informed by the Geotechnical Interpretative Reports (GIR) prepared at Tender design stage for the Awatoto and Pakowhai pump stations.

While the same broad geological units are present across the sites, the relative thicknesses, depths, continuity, and in-situ test responses vary spatially, reflecting local depositional processes and the influence of engineered fill and stopbank construction. Accordingly, the ground model presented herein is high level only and should be regarded as an interpretation that captures the expected range of ground conditions rather than a precise stratigraphic definition at each pump station.

The geotechnical interpretation and ground models will be reviewed and refined at Preliminary and Detailed design stages, following completion of all additional geotechnical investigations and laboratory testing. Any updated interpretation will be documented within updated Geotechnical Interpretative Reports to support the detailed design phase.

5.2.2 Fill (Unit 1)

Fill materials were encountered at both sites and are associated with historic construction activities, including stopbank works and pump station infrastructure. The fill is variable in composition and thickness and is inferred to be non-engineered.

Across both sites, the fill generally comprises a mixture of gravel and silt with subordinate sand and minor clay fractions. The gravel and sand fractions range from fine to coarse grained. The density and consistency of the fill varies spatially, ranging from loose to dense in granular materials and from soft to very stiff where finer-grained components are present. Moisture conditions within the fill are typically described as moist to wet.

Standard Penetration Testing undertaken within the fill indicates corrected SPT N values generally in the order of approximately 4 to 15 blows per 300 mm. Cone Penetration Test (CPT) response within the fill is highly variable, with qc values ranging from low to high (approximately 1 MPa to greater than 10 MPa), reflecting the heterogeneous and non-engineered nature of the material. Localised spikes in qc are associated with gravel-rich zones.

5.2.3 Heretaunga Formation Alluvium (Unit 2)

The underlying natural soils at both sites comprise alluvial deposits of the Heretaunga Formation, consistent with the published regional geology. These deposits have been subdivided into sub-units based on material type, density, consistency, and in-situ test response.

5.2.4 Soft Silt (Unit 2a)

This unit consists predominantly of silt with minor clay and sand fractions. The sand component is typically fine grained. The consistency of this unit is described as very soft to soft. Corrected SPT N values obtained within this unit are typically low, generally in the order of approximately 0 to 5 blows per 300 mm. CPT results indicate low cone resistance, with qc values generally less than 1 MPa and typically in the order of approximately 0.2 to 1 MPa. Moisture conditions are described as wet to saturated.

5.2.5 Very Loose to Loose Silty Sand (Unit 2b)

This unit comprises a major sand fraction with minor silt and trace clay, organics, and shell fragments locally present. The sand is generally fine to medium grained. The density of this unit ranges from very loose to loose, with corrected SPT N values typically in the range of approximately 1 to 7 blows per 300 mm. CPT results indicate low cone resistance, with qc values typically in the order of approximately 1 to 5 MPa. Moisture conditions are described as wet to saturated.

5.2.6 Medium Dense to Dense Sand/Silt (Unit 2c)

This unit consists of interbedded sand and silt as the dominant fractions, with trace shell fragments locally present. The sand fraction is predominantly fine grained. Granular soils within this unit are described as medium dense to dense, while finer-grained soils are described as soft to stiff. Corrected SPT N values generally range from approximately 15 to greater than 30 blows per 300 mm. CPT results indicate a marked increase in cone resistance relative to the overlying units, with qc values generally greater than 5 MPa and commonly in the order of approximately 5 to 15 MPa or higher, reflecting improved density and stiffness. Moisture conditions are described as wet.

5.2.7 Soft to Firm Sandy Silt (Unit 2d)

This unit comprises predominantly sandy silt with subordinate sand and minor clay fractions. The consistency of the unit ranges from soft to firm, and the material generally exhibits low plasticity. Corrected SPT N values are typically in the order of approximately 8 to 12 blows per 300 mm where measured. CPT results indicate intermediate cone resistance, with qc values typically in the order of approximately 1.5 to 4 MPa, reflecting strength greater than Unit 2a but lower than Unit 2c. Moisture conditions are described as wet.

5.3 Groundwater

5.3.1 Awatoto pump station

Based on recorded groundwater levels within boreholes and CPTs undertaken by WSP in 2024, and more recently by GHD in 2026, groundwater was encountered at shallow depths across the site. It is noted the groundwater level at the site will be sensitive to the water levels within the adjacent drainage channel. For the purposes of geotechnical assessment, a representative groundwater level of RL 0.0 m has been adopted for long term conditions. An elevated groundwater condition at the landside of the stopbank has been adopted at RL 0.77 m (which exceeds the existing ground level at the site), corresponding to the 1% AEP flood water level, to represent conservative flood-influenced groundwater conditions during extreme events.

5.3.2 Pakowhai pump station

Based on recorded groundwater levels within boreholes and CPTs undertaken by WSP in 2024, and more recently by GHD in 2026, groundwater was also encountered at shallow depths across the site, influenced. For the purposes of geotechnical assessment, a representative groundwater level of RL 0 m (approximately 2 mbgl) has been adopted for long term conditions. An elevated groundwater condition has been conservatively assumed to occur at ground surface, reflecting the 1% AEP flood water level of approximately RL 2.4 m (landside), which exceeds the existing ground level at the site.

5.4 Geotechnical design parameters

Geotechnical design parameters have been derived from interpretation of the site investigation data and geological ground models developed for the Awatoto and Pakowhai Pump Station sites. While the sites are located within the same regional geological setting and exhibit comparable alluvial stratigraphy, geotechnical parameters have been adopted on a site-specific basis to reflect differences in local ground conditions. The numerical values of all geotechnical design parameters for each site are presented in the respective Geotechnical Interpretative Reports and should be referred to for design purposes.

As detailed design progresses, the parameters will be reviewed and may be refined during detailed design, where necessary, to account for finalised load cases, construction methodology, and any additional geotechnical information obtained.

6. Design criteria

6.1 Design life

For the purposes of seismic design, all geotechnical elements relating to structures and soil-structures will have a design life of 50 years.

6.2 Importance level

All geotechnical and soil structures associated with the pump stations will be designed based on Importance Level 2 (IL2), as defined in AS/NZS1170.0.

6.3 PGA and earthquake magnitude

For the purposes of geotechnical design, the peak ground acceleration (PGA) and earthquake moment magnitude (Mw) have been established in accordance with MBIE Earthquake Geotechnical Engineering Practice Module 1 (MBIE, 2021). The adopted seismic parameters for the Serviceability Limit State (SLS) and Ultimate Limit State (ULS) earthquakes are summarised in Table 4, consistent with the requirements of the Principal's Requirements.

In addition to the SLS and ULS design events, an intermediate-level earthquake is considered in the design to reflect the performance objectives set out in the Principal's Requirements. This event has been nominally defined as a 1-in-100-year return period earthquake and represents a credible level of seismic shaking at which the pump station and associated structures are required to remain functional or readily repairable, without disproportionate damage or loss of service. The intermediate-level earthquake is therefore used to inform assessment of foundation performance, ground improvement requirements, and detailing to accommodate movement. The SLS and ULS events are retained to confirm acceptable performance at lower and extreme levels of shaking.

Table 3 Seismic design loading

Input parameter	Parameter value	Reference
Design life (seismic)	50 years	Geotechnical PR Section 5.4.2
Importance Level (IL)	2	Geotechnical PR Section 5.4.2 NZS1170.0 Table 3.2
Annual probability of exceedance	25 / 100 / 500 years	Principal's Requirements Section 3.5.1 NZS1170.0 Table 3.3
Design magnitude earthquake	6.4 / 6.7 / 7.1	MBIE Module 1 (2021) Appendix A
PGA (SLS/Intermediate/ULS)	0.12g / 0.24g / 0.58g	MBIE Module 1 (2021) Appendix A

6.4 Site subsoil class

Site subsoil class has been established in accordance with NZS 1170.5:2004. The sites are considered to be site subsoil Class D. This is based on the presence of isolated zones of silty sand and sand yielding SPT N < 6 blows. However, it must be noted that the derivation of PGAs for this site (Napier area) is established independently of site subsoil class (via MBIE Module 1/Cubrinovski et al 2021).

7. Design methods

7.1 General

Assessment of geotechnical elements of the works will be undertaken generally following methodologies outlined in Table 4 below.

Table 4 Methods of analysis – geotechnical design

Design consideration	Method
Liquefaction assessment	CPT based assessment using CLiq software applying the Boulanger and Idriss (2014) triggering method with fines correction following Robertson and Wride (1998). Use of Liquefaction Potential Index (LPI), Liquefaction Severity Number (LSN) for Performance Level classifications in accordance with MBIE/NZGS Earthquake Geotechnical Engineering Practice – Module 3: Identification, assessment and mitigation of liquefaction hazards to characterise likely surface manifestation and severity of liquefaction.
Liquefied shear strength and cyclic softening	Estimation of post-liquefaction residual shear strength using the Idriss and Boulanger (2008) method, adopting a representative S_u/σ'_v ratio. Assessment of cyclic softening potential in fine-grained soils using the methods of Idriss and Boulanger (2007) and Bray and Sancio (2006).
Liquefaction-induced settlement and lateral spreading	Estimation of free-field vertical settlement using the Zhang et al (2002) method implemented within CLiq software. Semi-empirical assessment of free-field lateral spreading using CLiq software based on the Zhang et al (2002) methodology for level ground with a free face.
Ground improvement	The design of displacement inclusions shall generally be in accordance with the guidance of Ashford et al. (2013), Reducing Seismic Risk to Highway Mobility: Assessment and Design of Pile Foundations Affected by Lateral Spreading, OTREC-RR-13-05 (2013). Determination of minimum ARR using empirical relationships from Barksdale & Bachus (1983) and Baez & Martin (1993). Establishment of target equivalent SPT N_{60} value using Youd et al. (2001) charts.
Shallow foundations	Designed using the methods, load factors and reduction factors outlined in B1/VM2 – Geotechnical design of foundations. Layered bearing capacity assessment for shallow foundations using derived soil parameters appropriate to the stress influence zone. Post-earthquake bearing capacity assessment using the Meyerhof and Hanna (1978) method with liquefied soil strengths. Settlement estimated using analytical modelling in Settle 3 software (Rocscience Inc.) with cross checks by hand (Boussinesq) and using CPe-IT software.
Structural seismic sliding displacement	Estimation of local sliding displacement of the pump station structures using a Newmark sliding block approach as described in NZTA Bridge Manual, using yield accelerations determined from Slope/W.
Soil-structure interaction	The pump station structure is assumed to behave as a relatively stiff element founded on deformable alluvial soils, and structural analyses will account for interaction between foundation stiffness, groundwater conditions, and imposed seismic demand. Where simplified Load Resistance Factor Design (LRFD) analysis is used for bearing, sliding and overturning and uplift, assumptions regarding rigidity, load transfer, and deformation compatibility will be clearly stated.
Slope stability	Two-dimensional limit equilibrium slope stability analysis using GeoStudio SLOPE/W with the Morgenstern-Price method. Seismic slope stability analysis using pseudo-static loading with non-liquefied soil strengths. Slope stability analysis using liquefied soil strengths to represent post-ULS earthquake conditions. Estimation of permanent seismic displacements using Newmark sliding block method as described in NZTA Bridge Manual, using yield accelerations determined from Slope/W.

Design consideration	Method
Embedded retaining walls	Analysis of embedded pile retaining walls using WALLAP software, including static, elevated groundwater, and seismic design cases. Global stability (if applicable) will be assessed using Slope/W (or similar).
Stopbank penetrations	Assessment of penetration-related failure modes including seepage, internal erosion, local stability. Design of filter collars undertaken in accordance with BOPRC Stopbank Design and Construction Guidelines v1.2 and CIRIA C731. Design of stopbank profile on a like-for-like basis to the consented design (by others).

7.2 Slope stability

Slope stability assessment will be undertaken to confirm that permanent slopes associated with the works achieve acceptable performance under static, elevated groundwater, seismic, and post-seismic conditions. Stability performance will be assessed using a combination of factor-of-safety-based checks and deformation-based assessment, recognising that under seismic loading some loss of stability margin and associated permanent displacement may occur without resulting in unacceptable performance.

Target factors of safety for the governing load cases are summarised in Table 6.

Table 5 Slope stability target factors of safety

Load case	Soil parameters	GWL	Loading conditions	Target FoS
Long-term static	Drained (effective stress)	Long-term / normal	HN traffic 12 kPa where applicable	1.5
Elevated groundwater	Drained (effective stress)	Elevated – following AEP 1/100 water levels	HN traffic 12 kPa where applicable	1.25
Seismic	Drained (effective stress)	Long-term / normal	No live load SLS / Intermediate / ULS PGA's	>1.0 or acceptable displacements
Post-seismic	Liquefied (and cyclic softened) soil strength where applicable	Long-term / normal	No live load	>1.0 or acceptable displacements

For seismic and post-seismic load cases, FoS $\geq$ 1.0 is adopted as a screening criterion. Where analyses indicate FoS < 1.0, slope performance will be assessed on the basis of estimated seismic-induced permanent displacements using appropriate deformation-based methods (e.g. Newmark sliding block analysis).

8. Construction verification

Construction verification will be undertaken to confirm that the ground conditions encountered, materials used, and construction outcomes are consistent with the assumptions, parameters, and risk mitigation measures set out in the Geotechnical Interpretative Reports. Verification activities shall be proportionate to the geotechnical risk and criticality of the works, with particular focus on ground improvement measures, foundations, retaining walls, and stopbank penetrations.

Construction verification for geotechnical elements of the works shall include:

- Verification of ground conditions exposed during excavation and foundation preparation against the design ground model and engineering soil units defined in the GIRs, including confirmation of bearing strata, groundwater conditions, and any variability relevant to performance.
- Post-installation CPT testing within treated ground improvement zones, undertaken to verify that the target improvement objectives defined in the GIRs have been achieved. CPT results will be used to confirm improvements in soil strength and stiffness parameters (including cone resistance and inferred equivalent SPT N_{60} values) relevant to liquefaction resistance, settlement performance, and foundation support. The extent, timing, and acceptance criteria for CPT verification testing will be confirmed during detailed design in accordance with the GIR assumptions and NZGS/MBIE guidance.
- Inspection and verification of earthworks and stopbank materials, including confirmation that material types, zoning, placement, and compaction are consistent with the assumptions adopted in the GIRs, the consented stopbank design, and the Bay of Plenty Regional Council Stopbank Design and Construction Guidelines.
- In situ density and moisture content testing of compacted fills and stopbank materials in accordance with Quality Assurance requirements.
- Inspection of stopbank penetrations and reinstatement works, including verification of filter collars and detailing intended to maintain stopbank integrity and performance.
- Targeted observation of geotechnically sensitive construction stages, including excavations within or adjacent to stopbanks, foundation subgrade preparation, placement of engineered fill, and reinstatement of stopbank sections on a like-for-like basis.

Where ground conditions, material properties, or construction outcomes differ materially from those assumed in the GIRs, the geotechnical design will be reviewed in accordance with the observational approach set out in the NZGS/MBIE Earthquake Geotechnical Engineering Practice Modules. Any required changes to design parameters, mitigation measures, or construction methodology will be documented, assessed, and submitted to the Engineer for acceptance prior to implementation.

Construction verification activities are intended to confirm consistency with design intent and do not replace the Contractor’s quality control obligations.

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