

Technical Report B

Geotechnical

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Geotechnical Assessment Report

Redvale Landfill Expansion

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Executive summary

Tonkin & Taylor Ltd (T+T) has been engaged by WM New Zealand (WMNZ) to provide technical assessments to understand the effects from a proposal to extend the use of Redvale Landfill, north of Auckland (the Project).

Redvale Landfill is currently scheduled to cease accepting waste in December 2028 and due to a delay in the authorisation and establishment of the replacement Auckland Regional Landfill (ARL) a transition plan is required. The proposal is to continue sending the current residual waste (around 600,000 tonnes per year) to Redvale Landfill, beyond the existing consent timeframe of December 2028. This would require new resource consents at Redvale Landfill.

Based on the available information the following summarises our conclusions with respect to the geotechnical design and construction of the proposed extensions to the Redvale Landfill:

A number of geotechnical and hydrogeological studies were carried out for the Redvale Landfill prior to 1992 to support the original resource consent application. Since that date a large number of geotechnical and related studies and reports have been prepared by T+T as the landfill has been progressively developed. Most recently 3 No. additional machine boreholes were carried out along the Landfill's northern boundary and 1 No. machine borehole within the proposed extension area to the south-west of the landfill, with piezometers installed in each borehole.

An extensive programme of geotechnical instrumentation has also been progressively installed since 1988 at the Redvale Landfill site, principally comprising piezometers, inclinometers and survey pins.

The results of the studies and investigations have been used to develop an Engineering Geology model for the Redvale Landfill site, which has been progressively revised and upgraded as the landfill development proceeded. The model shows the site to be underlain by approximately north-south trending (Northland Allochthon) Limestone bands, separated by Mudstones and Siltstones, and with (Waitemata Group) Sandstones and Conglomerates present across the south-west corner of the site and to the south and west of the landfill boundary.

The Eastern Property and the eastern two-thirds of the Western Property are underlain by sheared, generally NNE to SSW trending Northland Allochthon deposits. These are dominated by a prominent central (Mahurangi) Limestone mass aligned approximately along the Properties' common boundary, flanked to the west and east by sheared, often contorted, interbanded Mudstones and Siltstones.

Waitemata Group Sandstones and Siltstones form the bedrock geology to the south and west of the Redvale Landfill boundary. The inferred junction between those units and the Northland Allochthon materials is delineated by a steeply WSW dipping thrust fault, which runs approximately NNW to SSE across the south-western edge of the Western Property and extends southwards across the western edge of the proposed Class 2 extension. Along the thrust fault is a zone of Autoclastic Breccia, formed during the Allochthon emplacement process, with further, irregularly interbanded, sub-parallel bands of Limestone, Autoclastic Breccia and Waitemata Group extending south-westwards towards and beyond the Landfill boundary and the Waitemata Group country rock.

Prior to development and basegrade excavation, Pleistocene Sediments and/or Northland Allochthon residual soils formed the immediate surface materials over much of the study area. Low angle-shear planes are often present at the base of those Pleistocene Sediments and within the underlying weathered Northland Allochthon mudstones and siltstones.

Variable thicknesses and types, of engineered fill have also been placed over the parts of the development area since 1988 and further fill will be necessary to complete the works.

The hydrogeological regime and groundwater model for the overall Redvale Landfill site is discussed in the Hydrogeology Report which should be referred to for further detail.

Geotechnical design parameters have been assigned to the various geological units and fill materials based on geotechnical assessments and investigations to date.

All cut slopes around the perimeter of the current Landfill void (prior to waste placement) have been formed at 1V:3H, varying to about 1V:4H depending on the slopes' height and ground and groundwater conditions. The great majority of the basegrade cut slopes were in in-situ Northland Allochthon, with the exception of the upper portions of the highwall in the south-west corner of the Landfill which was formed in Waitemata Group materials. Fill slopes have also been formed at 1V:3H or flatter.

Pre-landfill development the original topography sloped down towards the north and north-west. To achieve sufficient airspace volumes a bund (fill embankment) termed the 'Phase 10 Bund' was constructed along the western half of the northern boundary.

The proposed works to create that additional airspace comprise two principal construction components, namely:

- 1 The current final design landfill ground profile varies but is everywhere less than 1V:3H. That ground profile will be steepened to a maximum grade of 1V:3H to allow placement of additional waste. Along the Eastern and Southern Boundaries bunds, varying from 2 m to 6 m high, will be constructed along the current landfill perimeter to allow the additional waste volumes to be achieved.
- 2 The landfill will be extended to the south-west of the current Landfill boundary, below the area of the now decommissioned Managed Fill and to the east of Sedimentation Pond 6. The extension will comprise the excavation and lining of a new landfill cell, to allow placement of Class 2 waste, and constructed in a similar manner to the earlier cells within the current landfill.

The design criteria for the stability of the proposed cut and fill for the Redvale Landfill have been based on the conditions proposed for the Auckland Regional Landfill (ARL) development. Design basis earthquake (DBE) and Maximum credible earthquake (MCE) peak ground accelerations of 0.19 g and 0.28 g respectively have been used for the long-term slope stability seismic analyses for the Redvale Landfill. The Class 2 extension development "temporary" slopes are likely to have a design life of between about 1 and 6 years, hence lower pga values have been used.

Slope stability analyses have been carried out along 10 No representative cross-sections through the proposed development. The results of the analyses show that:

- 1 Northern Boundary, Eastern Boundary and Southern Boundary:
 - The assessed stability of the proposed development final waste slope profile satisfies design criteria for design groundwater, elevated (high) groundwater and seismic loads.
- 2 Class 2 waste extension:
 - The proposed Class 2 extension basegrade slopes satisfy design criteria for design groundwater, elevated (high) groundwater and seismic loads.
 - The assessed stability of the proposed Class 2 extension waste slope profile satisfies design criteria for design groundwater, elevated (high) groundwater and seismic loads.

The above conclusions are based on stability analyses carried out assuming likely 'worst-case' material strength parameters and groundwater conditions. In practice material strengths could be greater than those assumed 'worst-case' values, over some or all of the development area. Stability FoS will therefore likely further increase beyond the assessed, already acceptable, values. Stability

analyses using the 'piezometric line with B-bar' approach were also carried out which concluded that generation of excess pore water pressures, as a result of the proposed extension works, is unlikely.

Investigations for the proposed perimeter bund along the Southern Boundary have identified Pleistocene Sediments locally up to 10.2 m depth, likely representing an infilled channel in the underlying Northland Allochthon paleo-topography. The extent of that feature, and possible design and construction options for the bund at that location, will need to be confirmed as part of detailed design. Provisionally options could include excavation and replacement with engineered fill, partial excavation/replacement, or founding the bund directly over the Pleistocene Sediments.

The proposed development works will be constructed to the requirements of the Technical Specification used for the current Landfill development, which documents the key design and construction requirements.

The lining system material will be selected Northland Allochthon Mudstone or Siltstone, as used successfully for all existing low-permeability liner at the Redvale Landfill. The material will be sourced either from excavations or from selected stockpile borrow areas. Based on the results of extensive permeability testing to date it is expected that such proposed lining system materials will readily be able to meet the required coefficient of permeability for the Class 2 extension lining system of less than 1×10^{-8} m/sec.

During construction the works should be regularly inspected and mapped by an experienced Engineering Geologist and/or Geotechnical Engineer, to confirm agreement with the design assumptions.

On the basis of the reviews and analyses set out in this Geotechnical Assessment Report we consider that, from a geotechnical perspective, the site is suitable for the construction of the proposed landfill extension works, comprising steepening the current final landfill profile by means of a waste overlay and extending the landfill's area to the south.

1 Introduction

1.1 Background

Tonkin & Taylor Ltd (T+T) has been engaged by WM New Zealand (WMNZ) to provide technical assessments to understand the effects from a proposal to extend the use of Redvale Landfill, north of Auckland (the Project). This report forms part of a suite of technical reports prepared for WMNZ for inclusion in a substantive application for resource consent under the Fast-Track Approvals Act 2024.

This report assesses the actual and potential geotechnical effects of the Project and is intended to support the Assessment of Environmental Effects (AEE).

1.2 Scope of work

The main objective of this Geotechnical Assessment Report is to confirm that:

- 1 The stability of the proposed final landfill ground profile, that is the waste overlay and perimeter bunds where present, is adequate.
- 2 The geotechnical design and construction procedures proposed for the landfill extension (understood to be for Class 2 waste) are appropriate.
- 3 The stability of the proposed revised Class 2 extension basegrade is adequate during construction.

The scope of this Geotechnical Assessment Report includes the following:

- 1 Review of the existing consented geotechnical design and construction procedures.
- 2 Discussion of the proposed landfill extension development.
- 3 Review of historic and recent geotechnical investigations and construction records, with discussion of geotechnical design parameters and development of a ground and groundwater model.
- 4 Slope stability analysis of representative cross-sections through the proposed landfill final ground profile.
- 5 Recommendations and conclusions with respect to the geotechnical aspects of the proposed landfill final ground profile and landfill extension development.

1.3 Site description

The Redvale Landfill is located in Dairy Flat, to the north of Auckland, approximately 9 km northwest of Albany and 1 km west of State Highway 1. Redvale Landfill, including the existing landfilling area and buffer zones, occupies approximately 88 ha in area. The landfill is divided into five main sub-areas for the purpose of describing the work within this application, known as the Northern, Eastern, Western, South-Western and South-Eastern Property.

The Landfill is accessed from the south-east along Landfill Access Road off Dairy Flat Highway, with Horseshoe Bush Road running along part of the northern boundary of the Landfill area. A tributary of the Rangitopuni Stream runs eastwards along the northern and eastern site boundaries towards the Dairy Flat Highway. A full description of the site is provided in the AEE prepared for the project.

1.4 Project description

Redvale Landfill is one of only two landfill sites in the Auckland region that can receive >350,000 tonnes of waste annually to service the approximately 1.5 million tonnes of waste that the Auckland

Region produces each year. Redvale Landfill is currently scheduled to cease accepting waste in December 2028 and due to a delay in obtaining the resource consents required for the establishment and operation of the replacement Auckland Regional Landfill (ARL), WMNZ considers a transition plan is required. To address this potential gap in critical landfill infrastructure for residual waste disposal in the Auckland region, WMNZ is proposing to extend the operation of Redvale to the end of 2036 and develop 4.56 Mm³ of additional airspace at the existing facility. This includes 3.13 Mm³ in additional Class 1 Landfill expansion on top of the existing Class 1 refuse placement area and 1.43 Mm³ of Class 2 Landfill extension. The proposed works include the following:

- **Class 1 landfill expansion:** placement of waste within the existing landfill footprint, involving the removal of final cover and steepening of the final cap slope to accommodate additional waste.
- **Class 2 footprint extension:** extension of the existing landfill to the southwest to accommodate disposal of waste meeting Class 2 landfill waste acceptance criteria (WAC).
- **Relocation of ancillary activities:** the relocation of ancillary activities to allow for the placement of waste.
- **Sediment Retention Ponds:** The removal and filling in of Sediment Retention Pond 6 and the Office Pond while enlarging Pond 1 to capture and treat the stormwater from these catchments.

A full description of the proposal is included in the AEE prepared for the project.

1.5 Previous and current studies

1.5.1 Previous studies

Several geotechnical, ecological and hydrogeological studies were carried out for the Redvale Landfill prior to 1992 to support the original resource consent application. Since that date many geotechnical and related studies and reports have been prepared by Tonkin & Taylor Ltd (T+T) as the landfill has been progressively developed. The general approach adopted during the landfill has been to prepare a Geotechnical Design Report to support each proposed construction activity and its associated construction drawings. That Report has then formed part of the documentation to be reviewed and approved by the appointed Peer Review Panel (PRP). The Reports were principally prepared for each individual landfill cell (Phase) but were also prepared for proposed stockpiles, sedimentation ponds and on-site structures.

Section 13 of this report lists what are considered to be relevant references, and of particular relevance is the Geotechnical Review Report¹ that was prepared as part of supporting documentation for the 2014 resource consent application. That report documented and summarised the works to that date, with presentation and discussion of the assessed geotechnical model for the landfill. That report, and geotechnical model, is still considered valid, albeit updated by the geotechnical works since that reporting date and has formed much of the basis for this current assessment and report.

1.5.2 Recent studies

Following completion of the 18 July 2024 Technical Memo,² 3 No. additional machine boreholes were carried out along the Landfill's northern boundary and 1 No. machine borehole within the proposed Class 2 waste extension area to the south-west of the landfill, see Engineering Geology

¹ Tonkin & Taylor Ltd : November 2013: Transpacific Waste Management. Redvale Landfill Consents Renewal. Geotechnical Review Report. Ref 29331.300

² Tonkin & Taylor Ltd : 18 July 2024 : Waste Management (NZ) Ltd. Redvale Landfill. Concept Geotechnical Stability Assessment, Technical Memo, v2.

Plans Figures D1 and D2 in Appendix D. In October 2025 15 No. rotary cored boreholes were carried out along the proposed alignment of the perimeter bunds. These investigations are discussed in more detail below in Section 5.3 and Section 9.3 respectively.

2 Site and general geology

2.1 The site

Prior to redevelopment as a landfill the site was used as a limestone (limerock) quarry by Redvale Lime Ltd. The quarry works were primarily in the central part of the overall landfill site with minor extraction from a smaller source in the western area. The landfill and quarrying operations co-existed from commencement of the landfill in 1992 to when Redvale Lime Ltd vacated the site in 2004.

Prior to the landfill (and quarry) development most of the site was low relief, sloping gently northwards and eastwards, and corresponding to the lower strength Northland Allochthon materials. Higher ground, generally corresponding to relatively more resistant Waitemata Group sediments, is present over the south-western part of the site, rising to a well-defined ridge to the south of the landfill boundary. A tributary of the Rangitopuni Stream runs eastwards along the northern and eastern site boundaries towards the Dairy Flat Highway.

2.2 Landfill development to date

The Landfill area is divided into two main sub-areas, defined as the East Property and the West Property, see Appendix B Key Plan Figure B1. The consented landfill area is defined by two lines, namely an inner Line A corresponding to the limit of placed waste, and an outer Line B corresponding to the outer edge of the landfill lining system, see Appendix D Engineering Geology Site Plans Figures D1 and D2. The East Property is about 36 ha in area with the design basegrade falling southwards from about RL 45 m to about RL 40 m, i.e. generally between about 15 m and 30 m below the original, pre-development ground level. The West Property is smaller in area, at about 23 ha, and with a higher design basegrade of about RL 70 m falling south-eastwards to about RL 57 m.

The landfill development has been carried out by progressive construction of individual landfill cells, referred to as Phases and numbered from Phase 1 to Phase 22, with some split into sub-phases, e.g. Phases 18A and 18B. The subgrade has been cut to design profile, with lining system placed to date over the entire consented Landfill area with the exception of a small area, referred to as Phase 21C(2) in the extreme south-west corner. In addition to the landfill cell (Phase) development per se other construction works have included temporary stockpiles, both inside and outside the landfilling area, sedimentation ponds and facilities such as the offices, workshop and the co-generation area.

In the late 1990s the area to the south of the central part of the Landfill's southern boundary was developed as part of the construction of Stockpile 4. The stockpile construction required substantial shear keys, to ensure adequate stability, with Sedimentation Pond 6 later constructed at the south-eastern toe of the formed slope. Subsequently, in the early 2010s, a Managed Fill site was developed over part of the earlier north haul road construction. The Managed Fill was designed and construction overseen by Soil & Rock Consultants Ltd (S&R) and included forebays constructed upstream of the existing Sedimentation Pond 6 downslope to the south-east³. The stockpiled fill was progressively used on site for daily and intermediate cover and has now been completely removed. The proposed Class 2 waste basegrade extension, see Section 4.2 below, will include the area of the former Managed Fill, to the west of Sedimentation Pond 6, and its general outline has been added to the Appendix B Key Plan, Figure B1.

The East Property basegrade was constructed entirely in cut with the exception of a short portion of the Phase 9 cell northern boundary, at the north-western corner of the Eastern Property. Over that

³ Soil & Rock Consultants : 2 September 2011: Geotechnical Investigation and Geometric Design, Proposed Managed Fill, Dairy Flat; Soil & Rock Consultants : 10 January 2013: Earthworks Completion Report Managed Fill-Site, Redvale Landfill

area, to achieve the required cell depth, and overall final landfill contours, a fill embankment (the 'Phase 9 Bund') up to about 6 m high was constructed. A similar approach was required along the entire northern boundary of the Western Property where the 'Phase 10 Bund' was constructed between 1992 and 1994. The Phase 10 Bund runs along the northern edge of the designated landfill area with the Landfill boundary, i.e. Line A, running along the crest of the Bund. Its height varies from about 5 m at the western end, increasing eastwards to a maximum of about 12 m relative to the original, pre-development ground profile.

In order to allow efficient development of the landfill it was necessary to construct a number of temporary stockpiles both within, and outside the landfilling area. Material from those stockpiles is being progressively utilised during the construction of the landfill including for bulk fill, liner construction, daily cover and the landfill final cover.

2.3 General geology

The published geological maps for the area,⁴ show the Redvale Landfill site to be underlain by Northland Allochthon deposits. The Northland Allochthon, (formerly known as the Onerahi Chaos Breccia), was initially deposited as marine sediments in a deep sedimentary basin. When subduction began beneath Northland in the early Miocene, those marine sediments along with parts of the ocean crust were emplaced as thrust sheets on top of existing rocks to form the Northland Allochthon. Waitemata Group sediments were deposited during the period of Allochthon emplacement and have been picked up and included within the Allochthon mass. The lateral displacement and thrusting endured by the materials during the Allochthon emplacement process has resulted in the generally extensive degree of shearing and deformation, particularly of the weaker units, and the overall structural complexity of the unit.

Published data⁵ indicates Northland Allochthon units below the Redvale Landfill site to comprise Mahurangi Limestone, (forming the resource for the earlier quarrying works), and Hukerenui Mudstone. East Coast Bays Formation Siltstones and Sandstones, part of the Waitemata Group, are identified to the south-west of the landfill site. Overlying the site is a variable thickness of Pleistocene Sediments with localised Recent Alluvium also probably originally present in stream channels and pond areas.

Prior to the current Landfill's development Pleistocene Sediments originally mantled the site as the immediate surface materials, but were progressively removed as the landfill was developed. They are typically represented by firm to stiff, locally very stiff to hard, light grey and light orange-grey and orange mottled, silty Clay with scattered rootlets and organic debris, locally including shattered tree branches, trunks and stumps.

Various types of engineered fill, as defined by the Technical Specification,⁶ have been placed as part of the Landfill development, principally comprising site-won materials derived from Northland Allochthon mudstones/siltstones and to a lesser extent limestone.

The geological units encountered at the Redvale Landfill are discussed in more detail in Appendix L.

⁴ Institute of Geological and Nuclear Sciences Ltd: 2001: Geology of Auckland, 1:250,000. Sheet 3; Kermode L : 1992 : Geology of the Auckland Urban Area. 1: 50,000. Sheet R11. Institute of Geological and Nuclear Sciences Ltd

⁵ Institute of Geological and Nuclear Sciences Ltd: 2001: Geology of Auckland, 1:250,000. Sheet 3; Kermode L : 1992 : Geology of the Auckland Urban Area. 1: 50,000. Sheet R11. Institute of Geological and Nuclear Sciences Ltd

⁶ Waste Management (NZ) Ltd : 1994 : Phase 10 Bund Toe Key As-Built Drawings

2.4 Geological model

The extensive geological and geotechnical data gathered at the Redvale Landfill site since the original resource consent process commenced in 1988 have enabled a comprehensive and robust geological model to be developed for the site, see Engineering Geological Plans, Figures D1 and D2.

The model has identified the Eastern Property and the eastern two-thirds of the Western Property to be underlain by sheared, generally NNE to SSW trending Northland Allochthon deposits. These are dominated by a prominent central (Mahurangi) Limestone mass aligned approximately along the Properties' common boundary, flanked to the west and east by sheared, often contorted, interbanded Mudstones and Siltstones.

Waitemata Group Sandstones and Siltstones form the bedrock geology to the south and west of the Redvale Landfill boundary. The inferred contact between those units and the Northland Allochthon materials is delineated by a steeply WSW dipping thrust fault which runs approximately NW to SE across the south-western edge of the Western Property and then NNE to SSE to the south of the Landfill. Along the thrust fault is a zone of Autoclastic Breccia, formed during the Allochthon emplacement process, with further, irregularly interbedded, sub-parallel bands of Limestone, Autoclastic Breccia and Waitemata Group extending south-westwards towards and beyond (south of) the Landfill boundary and the Waitemata Group country rock.

Pleistocene Sediments and/or residual soils derived from weathering of the Northland Allochthon mudstone/siltstone formed the immediate surface materials over much of the study area, which have mostly been stripped away as part of site development. Low-angle shear planes have been observed at the base of those Pleistocene Sediments and within the underlying weathered Northland Allochthon mudstones and siltstones.

3 Existing design

3.1 Background

Limestone had been quarried at the Redvale site since at least the 1960s resulting in an increasingly large excavation void. The site's inherent suitability for sanitary landfill development, i.e. essentially backfilling of the quarry void with waste, had been initially considered in 1986. In early-1988 Waste Management NZ Ltd identified the site as a possible sanitary landfill site and began an initial programme of site investigations and planning studies to confirm the site's general suitability.

A series of geotechnical investigations, comprising engineering geology mapping, boreholes and trenches, were carried out by various parties to provide relevant information for the consented landfill design to be developed. A geological model developed from those investigations identified the site to be essentially underlain by north-south trending Northland Allochthon Limestone bands, separated by Mudstones and Siltstones, and with Waitemata Group Sandstones and Conglomerates present across the south-west corner of the site.

Following hearings, consents were issued in 1992 for the development of the Redvale Landfill.

3.2 Geotechnical design and construction

The original landfill basegrade layout proposed cut (and fill) slopes at 1V:3H or flatter. Such profiles met slope stability design criteria and were also selected for constructability, i.e. to allow practical safe plant access. All cut slopes around the perimeter of the Landfill void have therefore generally been formed at 1V:3H, varying to about 1V:4H depending on the slopes' height and ground and groundwater conditions. The great majority of the basegrade cut slopes were in in-situ Northland Allochthon, with the exception of the upper portions of the highwall in the south-west corner of the Landfill, (the West Property's Southern Highwall), which was formed in Waitemata Group materials.

Pre-landfill development, the original topography generally sloped down towards the north and north-west. Within the West Property, to achieve sufficient airspace volumes, a bund (fill embankment), termed the 'Phase 10 Bund', was constructed along the western half of the northern boundary. That resulted in the lower part of the basegrade along that portion of the landfill boundary being in cut in-situ Northland Allochthon materials with the upper part formed on a fill embankment. As a result of instability during initial construction, namely translational sliding of Fill over Pleistocene Sediments and weathered Northland Allochthon, a substantial shear key was constructed below the bund to meet the slope stability design criteria, see Appendix D Figure D2. Construction along the eastern half of the northern boundary was more straightforward, however, with basegrade able to be formed principally in cut within in-situ Northland Allochthon materials.

Permeability tests carried out as part of the geotechnical investigations had identified in-situ ground permeabilities to be very low, and typically less than 1×10^{-9} m/sec over much of the site area. In view of the in-situ basegrade materials being assessed as already having such a low permeability, the landfill lining system for the Redvale Landfill to date has therefore been constructed as a clay liner formed of recompacted material with a permeability of less than 1×10^{-9} m/sec.

To supplement visual assessment of construction operations, Quality Control testing of structural fill and liner fill placement was carried out, comprising material density, moisture content, strength and permeability determinations on in-situ and laboratory samples. As each landfill cell was developed the basegrade geology was inspected and mapped by representatives of T+T and the Peer Review Panel (PRP) to ensure compliance with the Specification and consent conditions.

4 Proposed development

4.1 General

The objective of the proposed Landfill extension development is to create additional airspace (for waste placement) at Redvale landfill. The proposed works to create that additional airspace comprise two principal construction components, namely:

- 1 The current final design landfill ground profile varies but is everywhere less than 1V:3H. That ground profile will be steepened, to up to a maximum grade of 1V:3H to allow placement of additional waste within the existing landfill footprint. Along the Eastern and Southern boundaries, bunds varying from 2 m to 6 m high, will be constructed along the current landfill perimeter to allow the required additional waste volumes to be achieved.
- 2 The landfill will be extended to the south-west of the current Landfill boundary, below the area of the now decommissioned Managed Fill and to the west of Sedimentation Pond 6, which has an underlying shear key. Shear keys are also associated with construction of the North and South Haul Roads for Stockpile 4, within the proposed extension area.

The extension will comprise the excavation and lining of a new landfill cell, to allow placement of Class 2 waste, which will be constructed in a similar manner to the already constructed cells within the current landfill.

The selected option design is discussed in the Landfill Engineering Report,⁷ and plans showing the proposed landfill works including modified existing landfill (capped waste) design profile, and the landfill extension basegrade excavation profile for Class 2 landfill materials, are enclosed as Appendix B Figures B2 and B3 respectively.

4.2 Landfill ground profile

The principal components of the selected option are:

- 1 Northern Boundary:
 - Waste slopes along the landfill's northern boundary are proposed to be steepened to 1V:3H, with maximum elevations at RL 106.7 m.
 - The northern boundary of the landfill was constructed in the early to mid-1990s. Stability issues required the 'Phase 10 Bund' along the western half of the boundary to be underlain by a substantial shear key, as discussed in Section 3.2.
- 2 Eastern Boundary:
 - A 2 to 6 m high bund is proposed to be constructed along the eastern landfill boundary, with waste slopes similarly steepened to 1V:3H, and with maximum elevations at RL 106.7 m.
 - It has been assumed that the perimeter bunds will be either founded on competent material, or preferably keyed into the underlying Northland Allochthon, and will be constructed to underlie the proposed perimeter road.
- 3 Southern Boundary:
 - A 2 m high bund is proposed to be constructed along the eastern part of the landfill's southern boundary, with waste slopes similarly steepened to 1V:3H, and with maximum elevations at RL 106.7 m.

⁷ Tonkin & Taylor Ltd : June 2025: Redvale Landfill. Landfill Engineering Report. Ref 1092369.3000, v1

- It has been assumed that the perimeter bunds will be either founded on competent material, or preferably keyed into the underlying Northland Allochthon, and will be constructed to underlie the proposed perimeter road.
- 4 Class 2 Extension:
- This design Option proposes extending the landfill profile as a 'lobe', constructed southwards and south-eastwards, over the original stockpile ground profile, former Managed Fill and current Pond 6 areas.
 - The basegrade will be formed at 1V:3H, extending eastwards down to a cell floor at about RL 66 m, to the immediate east of the existing Sedimentation Pond 6.
 - The lining system proposed for the Class 2 extension will be 0.9 m thick compacted clay fill, the same as the currently consented system used for the all the existing Landfill's cells.
 - Following completion of the basegrade and lining system placement, waste slopes will be formed at up to 1V:3H, with maximum elevations at RL 106.7 m.

5 Geotechnical investigations

5.1 General

Extensive geotechnical investigations have been carried out at the Redvale Landfill site, and immediate environs, since pre-consenting assessment of the site began in 1988. Those investigations can be split into four principal stages, namely:

The following sections briefly review and discuss the investigations to date.

- 1 Historic (pre-2024) investigations, comprising:
 - Pre-1992 investigations by T+T and others
 - Post-1992 investigations by T+T
 - 2010 Investigations by S&R
- 2 Recent 2024 investigations by T+T

Investigations principally comprised machine boreholes and test pits/trenches. A more limited number of Cone Penetrometer Tests (CPT), hand-auger boreholes and large diameter drilled shafts were also carried out. Other sources of geotechnical information obtained as part of the ongoing landfill development included data from construction observations, engineering geology mapping and the Basegrade Inspection process required under the current consents.

Laboratory testing, principally triaxial strength testing, have also been carried out on selected samples to confirm geotechnical design parameters.

5.2 Historic (pre-2024) investigations

The historic (pre-2024) geotechnical investigations are discussed below in Appendix M.

5.3 Recent (2024) investigations

5.3.1 General

Additional geotechnical investigations were carried out in October 2024 to support the results of the concept study and as recommended in the 18 July 2024 Technical Memo⁸. The principal objective of those investigations was to clarify and confirm groundwater conditions along the western part of the landfill's Northern Boundary and within the proposed Class 2 waste extension. The scope of work comprised:

- 1 Installation of replacement piezometers in 3 No. machine boreholes along the Northern Boundary. The existing piezometers in that area of the site had been no longer functioning reliably after about 2010, and the primary objective of the replacement piezometers was therefore to confirm current groundwater levels for design.
- 2 There was only limited ground and groundwater information available for the proposed Class 2 extension area, which lies outside (south of) the main landfill boundary. A borehole, with piezometers, was installed in that area to allow ground and groundwater conditions to be confirmed.

The boreholes were drilled by DrillForce Ltd, with the work supervised on a visiting basis by a T+T Engineering Geologist who also logged and photographed the core. The borehole locations are

⁸ Tonkin & Taylor Ltd : 18 July 2024 : Waste Management (NZ) Ltd. Redvale Landfill. Concept Geotechnical Stability Assessment, Technical Memo, v2.

summarised in Table 5.1 below and have been added to the Engineering Geology Plan D2 in Appendix D, with the borehole logs enclosed in Appendix E.

5.3.2 Boreholes

Three additional boreholes (W135, W136 and W137) were carried out along the western half of the northern boundary and one borehole (W138) within the proposed Class 2 waste extension area.

- 1 Borehole W135 is towards the western end of the Northern Boundary Phase 10 Bund, and taken to 15.5 m depth (RL 64.20). A single vibrating wire piezometer (Pz 40) was installed in the base of the borehole, see Section 5.3.3 below.
- 2 Borehole W136 is in the central part of the Northern Boundary Phase 10 Bund, and taken to 15.5 m depth (RL 62.95). The borehole location was selected to confirm ground and groundwater conditions along Cross-section N1, the principal cross-section analysed through the Northern Boundary, and which was in turn based on Cross-section 19A-N1 from the T+T 2014 Phase 19A Geotechnical Assessment Report⁹. Shallow (Pz 41) and deep (Pz 42) vibrating wire piezometers were installed in the completed borehole.
- 3 Borehole W137 is towards the eastern end of the Northern Boundary Phase 10 Bund, and taken to 15.5 m depth (RL 73.35). A single vibrating wire piezometer (Pz 43) was installed in the base of the borehole, see Section 5.3.3 below.
- 4 Borehole W138 is located within the proposed Class 2 extension (formerly covered by a Managed Fill stockpile), to the south of the central part of the Landfill's southern boundary. There is limited historic geotechnical information for that area and some of it is contradictory. The borehole's objectives were to clarify ground and groundwater conditions below the proposed landfill extension. The borehole was taken to 15.5 m depth (RL 62.88) with shallow (Pz 44) and deep (Pz 45) vibrating wire piezometers installed in the completed borehole.

The boreholes were carried out using water flush rotary coring to recover HQ-size core for visual inspection. Borehole locations were proposed by T+T and, following site discussions between T+T, WMNZ and DrillForce, revised slightly to allow for practical access constraints. The as-drilled locations were subsequently surveyed by WMNZ, with coordinates shown on the borehole logs in Appendix E.

5.3.3 Piezometers

A single vibrating wire piezometer was installed in W135 and W137, with two piezometers (shallow and deep) installed in each of W136 and W138. The vibrating wire piezometers were made up and installed under the direct supervision of Geotechnics Ltd, who commissioned the instruments, took the baseline readings, and have continued regular monitoring readings since installation. The piezometers are grouted into position, with particular care taken for the installation of the double-piezometers in Boreholes W136 and W138, and protected by a lockable upstand. Installation details of the vibrating wire piezometers are included in Table 5.1.

⁹ Tonkin & Taylor Ltd : March 2014 : Transpacific Waste Management. Redvale Landfill Project, Phase 19A Development Geotechnical Assessment Report. Ref 24976.320

Table 5.1: Boreholes and vibrating wire piezometer installation details

Borehole				Piezometer		
ID No.	Location	Ground RL (m)	Depth (m)	ID No.	Tip depth (m)	Tip RL (m)
W135	Northern Boundary	79.70	15.50	Pz 40	15.00	64.70
W136	Northern Boundary	78.45	15.50	Pz 41	8.00	70.45
				Pz 42	15.00	63.45
W137	Northern Boundary	88.85	15.50	Pz 43	15.10	73.75
W138	Class 2 extension	78.38	15.50	Pz 44	7.10	71.28
				Pz 45	15.10	63.28

5.4 Laboratory testing

The majority of historic laboratory testing at Redvale focused on assessment of material strengths, in particular of the Northland Allochthon sheared Mudstone and Siltstone. Other more limited testing has included material classification and index property testing. The following summarises that historic laboratory testing carried out to date.

- 1 Strength testing
- 2 Triaxial effective stress strength testing with pore pressure measurements, (CUP tests), has been carried out primarily on samples of Northland Allochthon sheared Mudstone and Siltstone, but also on samples of compacted fill. The tests were primarily multi-stage test although a number of single stage tests were also carried out on selected samples.
 - In order to assess residual strengths of shear zones and fractures within the Northland Allochthon sheared Mudstone and Siltstone, direct shear tests and ring-shear tests were carried out on sheared material and on clayey gouge from shear zones.
 - Unconfined Compressive Strength tests were carried out on samples of Waitemata Group sandstone recovered from boreholes above the Western Property's Southern Highwall.
- 3 5 During the earlier stages of the Redvale Landfill development several material classification and index property tests were carried out of samples of Northland Allochthon sheared Mudstone and Siltstone, in order to assist in assessing their likely behaviour for construction. Those tests included Atterberg Limit and a single Activity Index determination test.

The results from the laboratory testing were used to help assign material properties to the various units, as discussed in Section 8 below and set out in Table 8.1.

6 Geotechnical monitoring

6.1 General

An extensive programme of geotechnical instrumentation has been progressively installed since 1988 at the Redvale Landfill site, principally comprising:

- 1 Piezometers: To measure in-situ groundwater levels, pressures and heads. Instrument types include both simple open standpipes (wells) and low-volume piezometers.
- 2 Inclinometers: Installed in boreholes to accurately measure the magnitude, and depth, of possible ground displacements.
- 3 Survey pins: Installed on cut or fill slopes to measure surface displacement vectors.

In addition to the above geotechnically focused instrumentation, information from other monitoring equipment at the landfill has also been used on occasion.

6.2 Piezometers

6.2.1 Historic instrument

Two different piezometer types have been installed historically, namely simple open standpipes, which require a significant water volume change to allow groundwater levels to be monitored, and low-volume piezometers which rely on sensors to measure pressure head and thereby only require very small changes in water volumes record head changes:

- 1 Simple open standpipe piezometers or “wells”:
These generally comprise a plastic tube, with a slotted section at its base surrounded by a filter response zone, with the remaining length backfilled with arisings and/or bentonite cement grout. These are the commonest piezometer type and the W-series wells, installed by T+T, fall into this category.
- 2 Low-volume piezometers:
These essentially measure groundwater pressures, and hence piezometric heads, using remotely read sensors. The following types have been installed to date at the landfill:
 - Pneumatic piezometers.
 - Vibrating wire piezometers.
 - Remote data loggers. These instruments allow either delayed or real-time monitoring of groundwater pressures, and can also be retro-fitted to (W-series) standpipes.
- 3 7 All the currently functioning low-volume piezometers at the Redvale Landfill site are instruments installed by T+T since 1992 and are referred to as Pz-series instruments. Most have been installed directly in W-series boreholes but a number were installed in filter zones during the construction works.

Not all the piezometers historically installed at the Redvale Landfill site are still functioning. Some have been destroyed during the ongoing landfill cell development works and several of the low-volume piezometers are either no longer functioning or are recording erratic, or clearly anomalous data. Some of those assessed non-functioning low-volume piezometers have been replaced by the instruments installed as part of the recent October 2024 investigation works.

A comprehensive data spreadsheet of all piezometer installations and groundwater monitoring readings since their installation. The locations of all M-series, P-series, WL-series, and T+T Pz-series piezometers (including those recently installed in October 2024) and W-series standpipes (wells) are

shown on the Engineering Geology Plans, Figures D1 and D2 and on Monitoring Location Plan Figure F3.

6.2.2 Recent (October 2024) instrumentation

To assist in clarifying ground and groundwater conditions for the proposed Landfill extension design, four machine boreholes were carried out in October 2024, with vibrating wire piezometers installed in each borehole. Three boreholes were located along the western part of the Northern Boundary and one within the Class 2 extension (the former Managed Fill area). The borehole and piezometer locations are summarised in Table 5.1 above and are shown on the Engineering Geology Plans, Figures D1 and D2 in Appendix D, with the borehole logs enclosed in Appendix E. The piezometers' installation details are summarised in Table 5.1 and discussed in Section 5.3.3 above.

6.3 Inclinerometers

Eighteen inclinometers were installed at Redvale Landfill, during the construction works, see locations on Engineering Geology Plans Figures D1 and D2, Appendix D. The instruments were generally installed to monitor particular construction activities, and all have now either been destroyed or damaged during the course of the construction works, or have been decommissioned.

6.4 Survey pins

Survey pins were installed at a number of locations as part of the Redvale Landfill development works, principally for historic monitoring of a number of temporary slopes.

7 Geological and groundwater assessment

7.1 General

The general geology of the Redvale Landfill site is discussed in Section 2.3 above. Based on the available geotechnical information, the following Sections discuss and summarise the expected ground and groundwater conditions around the portions of the Landfill boundary affected by the proposed development, and for the proposed Class 2 waste extension to the south of the landfill.

7.2 Geological assessment

7.2.1 Northern boundary

7.2.1.1 Western half

As discussed in Section 3.2 above, to allow sufficient airspace to be obtained an embankment fill, termed the 'Phase 10 Bund', was constructed over existing ground along the western half of the Northern Boundary. The 'Phase 10 Bund' comprises layers of engineered fill constructed as an embankment over the Pleistocene Sediments and underlying weathered Northland Allochthon. The result is a composite basegrade profile slope, with the lower portion in Northland Allochthon Mudstone, predominantly Unit N7, or Limestone Unit N1, and the upper portion formed of Type 1C and Type 3 engineered Fill.

As a result of instability, comprising translational sliding of fill materials over the underlying Pleistocene Sediments, a substantial shear key formed of Type 1C fill was constructed along the northern boundary of the Bund. As-built records¹⁰ of the 'Phase 10 Bund' and shear key construction are enclosed in Appendix H with the shear key location also shown on the Engineering Geology Plan, Figure D2, in Appendix D.

Three machine boreholes, W135, W136 and W137 were recently drilled along this portion of the Northern Boundary and through the 'Phase 10 Bund'. All the boreholes encountered a similar profile comprising:

- 1 Engineered Fill: Between 5.1 m and 10.8 m of layered, stiff to very stiff silty Clay with variable minor gravel content and representing the Type 3 Fill forming the 'Phase 10 Bund' embankment; over
- 2 Pleistocene Sediments: Between 3.4 m and 4.7 m of typically stiff to very stiff silty Clay with traces of organic debris; over
- 3 Northland Allochthon: Up to 0.8 m to 5.7 m penetrated, and comprising dark grey to greenish grey, extremely weak to very weak unweathered Mudstone (Unit N7), with pervasively sheared Limestone (Unit N1) encountered at the base of W135.

The ground conditions encountered in the three recent boreholes were generally as expected and match historic investigation data and construction records.

7.2.1.2 Eastern half

Due to more favourable topography the requirement for embankment fill decreased eastwards along the Northern Boundary and basegrade for the eastern half of the boundary was therefore able to be formed principally within in-situ materials. Geotechnical investigations and construction records show that the basegrade was formed entirely in cut, with a band of Northland Allochthon

¹⁰ Waste Management (NZ) Ltd : 1994 : Phase 10 Bund Toe Key As-Built Drawings

Limestone (Unit N1) separating predominately Northland Allochthon Unit N7 Mudstones and Unit N3/N5 siltstones, see Engineering Geology Plan Figure D1 in Appendix D.

7.2.2 Eastern boundary

Ground conditions along the Eastern Boundary comprise Northland Allochthon materials with Unit N8 Mudstone encountered at basegrade in the northern third, Unit N4 Mudstone in the central third and Unit N6 Siltstone in the southern third of the boundary. Limited investigations to the immediate east of the Landfill boundary suggest a further generally NNE – SSW trending band of Northland Allochthon Limestone (Unit N1) within about 20 m to 50 m of the landfill boundary, see Engineering Geology Plan Figure D1 in Appendix D.

7.2.3 Southern boundary

Similar to the Eastern Boundary, ground conditions along the portion of the Southern Boundary affected by the proposed landfill reprofiling works are assessed to be a mixture of Northland Allochthon Unit N7 Mudstone and Unit N2 and N6 Siltstone. Recent investigations for the proposed perimeter bund have identified Fill overlying Pleistocene Sediments, with the latter locally up to 10.2 m depth and likely representing an infilled channel in the underlying Northland Allochthon paleo-topography.

7.2.4 Class 2 extension

Located outside (south of) the main landfill area per se, only limited historic geotechnical investigations were carried out in this area but available information indicates a Thrust Fault, separating Northland Allochthon and Waitemata Group materials, runs obliquely NNW to SSE across the western part of the proposed Class 2 waste landfill extension area, see Engineering Geology Plan Figure D2 in Appendix D.

Ground conditions identified in Borehole W138 generally match those expected, i.e. Fill overlying in-situ Pleistocene Sediments and Northland Allochthon. The thickness of Fill, at about 6.3 m, is somewhat greater than expected however and is likely to be, at least in part, remaining material from the earlier Managed Fill stockpile works. Underlying the Fill is about 1.5 m of Pleistocene Sediments represented by stiff silty Clay with variable organic content. The borehole confirmed the basal lithology, below the Pleistocene Sediments, as Northland Allochthon, penetrating up to 7.6 m of moderately weathered becoming unweathered, dark grey Mudstone, (likely Unit N7/N8). The borehole, by encountering Northland Allochthon bedrock, rather than Waitemata Group, supports the Engineering Geology Plan mapping interpretation in Figure D2. That shows the Thrust Fault, separating Northland Allochthon and Waitemata Group materials, to be towards the western boundary of the proposed Class 2 landfill extension area.

7.2.5 Discussion

The four additional boreholes recently completed along the landfill's Northern Boundary, and within the Class 2 extension area, all encountered ground conditions matching those expected from previous geotechnical studies and engineering geology mapping, i.e. fill overlying in-situ Pleistocene Sediments and Northland Allochthon.

7.3 Groundwater assessment

7.3.1 General

The existing groundwater data is historic and many of the existing piezometers are no longer functional, either due to their age or having been destroyed by construction activities. During and

following construction, in the early 1990s, of the Phase 10 Bund along the Northern Boundary, a number of monitoring wells and vibrating wire and pneumatic piezometers were installed. However, many of those existing piezometers are no longer extant, or are giving what are considered anomalous groundwater data. In the area of the proposed Class 2 waste extension, which lies outside the current consented Landfill area, there was limited groundwater information available.

To confirm the historic groundwater data along the Northern Boundary Phase 10 Bund, Boreholes W135, W136 and W137 were recently drilled with multiple replacement vibrating wire piezometers installed. Similarly, Borehole W138 was carried out in the proposed Class 2 waste landfill extension area primarily to allow installation of vibrating wire piezometers to confirm the groundwater conditions and its potential impact on the proposed basegrade excavation works.

The vibrating wire piezometers installed in those four recent boreholes have allowed current groundwater conditions to be assessed and compared to historic data.

7.3.2 Groundwater observation data

7.3.2.1 Historic data

A comprehensive data spreadsheet of all piezometer installations and groundwater monitoring readings since their installation is maintained by WMNZ. The locations of all M-series, P-series, WL-series, and T+T Pz-series piezometers (including those recently installed in October 2024) and W-series standpipes (wells) are shown on the Engineering Geology Plans, Figures D1 and D2.

7.3.2.2 Recent data

Table 7.1 below summarises the vibrating wire piezometer monitoring carried out by Geotechnics Ltd to date for the six piezometers installed recently in October 2024. The Table identifies what are appropriate design groundwater levels for those piezometer locations and compares the current levels with the available historic data. The Geotechnics record sheets are enclosed in Appendix F and a plot of measured piezometric Reduced Level against time is presented as Figure F1 in Appendix F.

Table 7.1: Summary of vibrating wire piezometer data

Borehole ID	Piezometer			
	ID	Measured piezometric levels		
		GW range RL (m) (23/10/2024 to 05/01/26)	Assumed design GW RL (m)	Depth (m) GW below existing ground level
W135	Pz 40	68.27 to 69.40	68.8	10.9
W136	Pz 41 (shallow)	72.48 to 74.02	73.0	5.5
	Pz 42 (deep)	72.17 to 73.66	73.0	5.5
W137	Pz 43	76.76 to 78.19	77.0	11.9
W138	Pz 44 (shallow)	72.95 to 73.69	73.5	4.9
	Pz 45 (deep)	72.09 to 73.12	72.5	5.9

Considering the data summarised in Table 7.1 and shown on Figure F1:

- 1 Since the piezometers were installed in October 2024, all instruments have shown a trend of falling measured piezometric levels for about the initial 3 to 4 months, likely a reflection of the very slow depressurisation of the ground following the effects of the drilling fluid. Since that date measured levels have been very steady with the possible exception of Pz 44 (the shallow piezometer within BH W138) which showed a gradual rise in measured level of about 0.8 m in

mid-2025, remaining relatively steady thereafter. That piezometer tip is within the surface Fill materials however and measured groundwater levels may therefore be being affected by the increasing seasonal (winter) rainfall. Based on the measured range an assumed groundwater level has been defined for each piezometer, as shown in Table 7.1.

- 2 Briefly reviewing the current piezometric data and comparing to historic data at each borehole location:
 - Borehole W135: Pz 40:
 - o There are no historic boreholes/piezometers near Borehole W135.
 - o Pz 7, about 45 m to the north-east, shows historic groundwater levels similar to those recorded for the current Pz 40.
 - o However W7, about 50 m north, and Pz 8 about 40 m to the north-west, showed levels typically lower and higher respectively compared to Pz 40.
 - Borehole W136: Pz 41 and Pz 42:
 - o The nearest historic piezometers to Pz 41/Pz 42 are Pz 16 and Pz 39, both about 70 m east and north-east respectively, and considered too distant to allow a meaningful comparison.
 - o The assumed design groundwater levels for Pz 41 and Pz 42 match well with the groundwater profile assumed earlier along Cross-section N1, and which was based on Cross-section 19A-N1 from the T+T 2014 Phase 19A Geotechnical Assessment Report¹¹.
 - o The shallow and deep piezometers show similar measured groundwater levels, confirming a hydrostatic groundwater profile and, in particular, that groundwater is not “perched” in the Fill overlying the in-situ Pleistocene sediments and Northland Allochthon.
 - Borehole W137: Pz 43:
 - o Pz 4 and Pz 37, both located about 20 m to 25 m north-east of Pz 43, show historic measured groundwater levels comparable to those in Pz 43.
 - Borehole W138: Pz 44 and Pz 45:
 - o Existing historical groundwater data in the vicinity of the Class 2 waste extension area is limited and contradictory.
 - o The assumed design groundwater levels for Pz 44 (shallow, in the Pleistocene sediments) and Pz 45 (deep, in the Northland Allochthon) are RL 73.5 m and RL 72.5 m respectively, i.e. about 1 m difference. Those ground water levels, assuming hydraulic connectivity,¹² therefore indicate groundwater in the base of the Fill.
 - o Assuming a general northerly dipping/flowing groundwater regime¹³ this suggests that there is hydraulic connectivity between the two piezometers (both in in-situ materials) but with a slight downwards hydraulic gradient (drainage).
 - o The current recorded groundwater level vs the current design basegrade levels therefore suggest groundwater heads of up to 10 m above the approximate RL 65 m proposed base of the Class 2 waste extension excavation. This is

¹¹ Tonkin & Taylor Ltd : March 2014 : Transpacific Waste Management. Redvale Landfill Project, Phase 19A Development Geotechnical Assessment Report. Ref 24976.320

¹² Tonkin & Taylor Ltd : June 2025: Redvale Landfill. Hydrogeological Assessment Report. Ref 1092369.6000, v1

¹³ Tonkin & Taylor Ltd : June 2025: Redvale Landfill. Hydrogeological Assessment Report. Ref 1092369.6000, v1

discussed further in Section 7.3.2.3 below and see also the Hydrogeological Assessment Report¹⁴.

7.3.2.3 Discussion

The piezometer monitoring to date indicates that:

- 1 Groundwater conditions along the Phase 10 Bund portion of Northern Boundary are considered generally similar to those recorded historically and assumed for both the previous phases of design and for the analyses considered in the earlier Summary Technical Memo. The monitoring data also confirms a hydrostatic groundwater profile and that groundwater is not “perched” in the Fill overlying the in-situ Pleistocene sediments and Northland Allochthon.
- 2 Within the proposed Class 2 extension area, measured piezometric levels are at about RL 73.5 m to RL 72.5 m corresponding to groundwater within the base of the fill, assuming hydraulic connectivity. This piezometric data indicates groundwater heads of up to 8 m at the lowest point of the proposed Class 2 extension basegrade. However, the following points should be noted:
 - Site experience has shown the Pleistocene Sediments, and particularly the underlying Northland Allochthon mudstone/siltstones, are often found to be effectively dry when excavating.
 - The location of Borehole W138 with Pz 44/Pz 45 is approximately 75 m north-west (upslope) of the proposed Class 2 waste extension’s RL 65 m lowest basegrade level. The measured groundwater levels, of about RL 72 m, correspond to a design basegrade level at the borehole/piezometer location.
 - The existing ground profile generally falls south-eastwards, towards Sedimentation Pond 6 (and the proposed lowest point of the Class 2 extension basegrade) and it is expected that the existing groundwater profile will follow a similar profile, i.e. flowing down eastwards towards the low ground corresponding to Sedimentation Pond 6 and the workshop area further to the east.
- 3 Such a view is supported by the base of Sedimentation Pond 6 (which is located over the approximate base area of the proposed Class 2 extension) and the adjacent workshop area being below RL 65 m, and with no known issues with groundwater seepages. In addition, it is understood that construction of Sedimentation Pond 6B, and shear key works, immediately west of Sedimentation Pond 6, included excavations to RL 64 m or lower. No issues with groundwater seepages or inflows into the excavations were encountered during construction.

In summary therefore, the recent investigations and piezometer monitoring suggest a groundwater table extending into the base of the fill and flowing down eastwards towards the low ground of Sedimentation Pond 6 and the adjacent workshop area.

7.3.3 Groundwater model assumed for design

7.3.3.1 General

Groundwater assessment during the development of the Redvale Landfill to date has focused on assessing the likely groundwater conditions as they relate to that particular phase of development, for example the design and construction of the next planned cell. A more general assessment of groundwater conditions, from a geotechnical perspective is presented in the 2014 Geotechnical

¹⁴ Tonkin & Taylor Ltd : June 2025: Redvale Landfill. Hydrogeological Assessment Report. Ref 1092369.6000, v1

Review Report,¹⁵ with a fuller assessment of the groundwater regime set out in the Hydrogeological Assessment Report¹⁶.

For the purposes of the progressive design and construction of the ongoing landfill development, the following has been generally assumed for design purposes, and is considered applicable to the current proposed development:

- 1 The tight, low persistence defects within the mudstone/siltstone rock mass and the high proportion of smectite clay minerals result in an assessed low permeability for those materials.
- 2 The laboratory test data, including results of triaxial strength testing and compaction monitoring, suggest that the near-surface intact Northland Allochthon mudstone/siltstone may not be fully saturated. Such an observation is considered to be a reflection of the high smectite content reducing the potential for migration of groundwater into the intact rock.
- 3 The distribution and the general decrease in flows with time observed from surface seepages and horizontal drainage drillholes suggest that groundwater is effectively in perched zones or “pockets”, the boundaries of which are formed by discontinuities such as shear zones, joints, bedding and changes in lithology. When bulk excavation or a drillhole breaches the boundaries of such a pocket, the groundwater contained within it is drained. As groundwater drains from the pocket flow rates decrease.
- 4 The permeability of the Northland Allochthon limestone fractured rock mass is generally greater than for any adjacent finer grained materials such as mudstone or siltstone.
- 5 Waitemata Group materials have only been encountered to date in the extreme south-west corner of the current Landfill development where they have been observed to be dry, with no evidence of seepages or perched groundwater. Such an interpretation generally agrees with observations of such materials within the wider Auckland region, indicating permeabilities likely comparable to the Northland Allochthon limestone.
- 6 The progressive groundwater assessments have generally assumed a hydrostatic groundwater profile, and a simple unconfined aquifer with a hydraulic gradient flowing towards the existing lower ground, i.e. generally the then base of the landfill (and earlier quarry) void. This aspect is discussed in greater detail in the Hydrogeological Assessment Report¹⁷.

7.3.3.2 Design assumptions

For the purposes of this assessment the following design assumptions have been made with respect to groundwater conditions for the proposed development:

- 1 Northern Boundary: Western half
For the western half of the Northern Boundary, containing the ‘Phase 10 Bund’, the following groundwater conditions have been assumed:
 - Design case: Groundwater level generally assumed at the top of the Northland Allochthon mudstone.
 - Elevated case: Groundwater level generally assumed at 1 m below existing ground level to the north of the boundary across the bund.
- 2 Northern Boundary: Eastern half; Eastern Boundary and Southern Boundary
For the eastern half of the Northern Boundary, the Eastern Boundary and the Southern Boundary, the following groundwater conditions have been assumed:

¹⁵ Tonkin & Taylor Ltd : November 2013: Transpacific Waste Management. Redvale Landfill Consents Renewal. Geotechnical Review Report. Ref 29331.300

¹⁶ Tonkin & Taylor Ltd : June 2025: Redvale Landfill. Hydrogeological Assessment Report. Ref 1092369.6000, v1

¹⁷ Tonkin & Taylor Ltd : June 2025: Redvale Landfill. Hydrogeological Assessment Report. Ref 1092369.6000, v1

- Design case: 3 m below existing ground level.
 - Elevated case: 1 m below existing ground level.
- 3 Class 2 extension
- Design case: Groundwater level generally assumed based on the piezometric data from Pz 44 and Pz 45, installed in Borehole W138.
 - Elevated case: Groundwater level generally assumed as varying up to 10 m above the design case.

Where appropriate, groundwater conditions are discussed in more detail below for each cross-section.

8 Geotechnical design parameters

8.1 General

The geotechnical design parameters, in particular the assessed strengths of both intact, (in-situ), materials and of compacted soils, (i.e. fill and lining system), have been progressively assessed as the landfill has been developed and additional data has become available.

The material properties of the in-situ and compacted soils, used in the stability analyses discussed in this memo, have been assessed from the results of previous laboratory testing, the performance of fill on site, previous geotechnical back-analyses, and the geological assessment of the behaviour of the jointed rock mass.

8.2 Design shear strength values

The assessed design shear strength values used in the Landfill extension slope stability analyses, both for in-situ and compacted soils, are summarised in Table 8.1 below.

Table 8.1: Design shear strength values

Geological unit	Unit weight (kN/m ³)	Effective cohesion (kN/m ²)	Effective friction Angle (°)
Landfill Refuse : used for landfill development to date	15	0	30
Landfill Refuse : new waste	12	5	30
Landfill Refuse : old (existing) waste	15	9	22
Type 1C compacted Mudstone (structural fill)	19	5	35
Type 3A compacted Mudstone (fill)	19	5	30
Pleistocene sediments : general southern boundary	18	5	20
	18	7	32
Mudstone (lower-bound sheared) – e.g. Type N7	20	6	22
Mudstone (failure plane) - general	20	0	10 - 14
Mudstone/Siltstone (isotropic) – e.g. Type N3, N5 or N6	20	28	14
Mudstone (anisotropic) : perpendicular to foliation : parallel to foliation	20	6 (max)	22 (max)
	20	0 (min)	14(min)
Limestone : Massive (quarry floor) : Fractured (highwall)	19	20	40
	19	15	35
Waitemata Group : Pre-NA Emplacement ~ perpendicular to foliation/banding ~ parallel to foliation/banding	20	100	40
	20	10	35
Waitemata Group: Post-NA Emplacement	20	5	32

Points to note in respect of the above design parameters include:

- 1 The landfill refuse (waste) parameters assumed to date for the design of the current landfill are relatively conservative. For the purposes of this assessment, design parameters have also been assessed for the proposed new waste, corresponding to short term conditions, and for the old (existing) waste some of which has been in place for over 30 years. The parameters have been assessed from published literature and from values used for other similar recent

landfill developments, such as the proposed Tirohia Landfill and the proposed Auckland Regional Landfill (ARL).

- 2 The mudstone Unit N7, as encountered within the study area, has a pervasive, but discontinuous, sheared fabric and has therefore been treated as sheared when assigning strength parameters. The degree of shearing and disturbance varies however and therefore lower-bound and upper-bound sheared values have been applied to the mudstone. Generally it is considered that the upper-bound sheared value is appropriate, except when near limestone contacts when the use of lower bound sheared values is considered more appropriate.
- 3 The shear strength parameters for failure surface/singular shears within the Mudstone rock mass have been derived from back analysis of historic movements at the Redvale Landfill. The assessed maximum value of 14° is considered appropriate as it makes an allowance of 3° to 4° for the “waviness” along potential failure surfaces, an aspect which cannot be satisfactorily measured by laboratory tests. Low-angle shear planes have been identified at the junction of the Northland Allochthon materials and the overlying Pleistocene Sediments, and these have been assigned similar properties as for the similar shear planes within the Mudstone rock mass
- 4 The strengths of the Waitemata Group sediments have been assessed based on visual inspection of the materials and extensive experience by T+T and others of similar materials within the Auckland region.

8.3 Landfill lining system fill

The low-permeability lining system for Redvale Landfill is specified to be either Type 1A or Type 1B material,¹⁸ with a compacted Coefficient of Permeability of not greater than 1×10^{-9} m/sec. It is understood that the compacted low-permeability liner for the Class 2 waste extension will be allowed to have a slightly higher Coefficient of Permeability of not greater than 1×10^{-8} m/ sec¹⁹.

To date, and also expected to apply to the Class 2 waste extension, all low-permeability lining system placed has been Type 1A fill comprising compacted Northland Allochthon Mudstone and Siltstone sourced from on-site borrow areas. Such fill materials, placed in compliance with the Technical Specification²⁰ and QA/QC procedures have always been readily able to achieve the specified permeability value.

No laboratory tests have been carried out on the proposed Landfill extension area borrow materials but, based on test results to date and the assumption that similar mudstone materials will be used, it is anticipated that the required permeability will be achieved. As part of the Specified QC procedures core-cutter samples of the compacted liner will be taken and laboratory tested to confirm compliance with the specified permeability value.

8.4 Landfill refuse

Redvale Landfill accepts a wide range of refuse ranging from domestic refuse to demolition debris to bulk fill. The material parameters adopted for the refuse are based on inspection and assessment of the performance of typical refuse at the landfill. Published literature,²¹ suggests a wide range of strengths for compacted refuse with effective cohesion values of 5 kN/m^2 or greater indicated for newly placed refuse, potentially increasing further for older materials. In a similar manner, published

¹⁸ Tonkin & Taylor Ltd : March 2015 : Specification. Redvale Landfill General Construction. Ref 30089.

¹⁹ Tonkin & Taylor Ltd : June 2025: Redvale Landfill. Landfill Engineering Report. Ref 1092369.3000, v1

²⁰ Tonkin & Taylor Ltd : March 2015 : Specification. Redvale Landfill General Construction. Ref 30089.

²¹ Singh S and B Murphy : 1990 : Evaluation of the Stability of Sanitary Landfills. Geotechnics of Waste Fills – Theory and Practice, ASTM STP 1070, pp 240 to 258

literature indicates a wide range of refuse unit weights, with the assessed value of 15 kN/m³ at the upper end of typical values,²² and therefore conservative in respect of stability analyses. Assessed material properties for the waste are given in Table 8.1 above.

9 Engineering considerations

9.1 General

The assessed stability of the proposed steepened final landfill (waste) slope profiles and of the proposed basegrade slopes for the Class 2 waste extension are discussed in this section.

It should be noted that the recommendations and opinions given in this Report are based upon data from readily available investigations such as boreholes, test pits, drilled shafts, mapping of surface features and construction observations. Inferences about the nature and continuity of ground conditions between investigation points have been made but cannot be guaranteed.

9.2 Design criteria

9.2.1 Stability design criteria

The engineering design and construction aspects considered in this report primarily comprise the assessment of the stability of the final landfill (waste capping) profile and of the basegrade excavation for the Class 2 waste extension below the former Managed Fill area. The following cases have been considered:

- 1 End-of-Construction Case: The effects on stability due to the construction of the proposed basegrade for the Landfill extension below the former Managed Fill area, i.e. prior to placement of Refuse fill.
- 2 Long-Term Case: The stability of the completed landfill following the placement of Refuse to the revised levels and profiles.

The slope stability analyses have been carried out on selected cross-sections through the Northern Boundary, Eastern Boundary, Southern Boundary and the landfill extension below the Class 2 area, as shown on Figures B2 and B3, in Appendix B.

The design criteria for the stability of the proposed cut and fill for the Redvale Landfill have been based on the conditions used for the Kate Valley, Canterbury Landfill and proposed for the Auckland Regional Landfill (ARL) development. For various defined design scenarios, recommended minimum design Factors of Safety (FoS) under static loading, and maximum displacements under seismic loading, are presented in Table 9.1 below:

Table 9.1: Stability design criteria

Design Scenario	Minimum design FoS (Static) Maximum displacement (Seismic)
Static Loading	
End of Construction, ('Design' GWL)	1.3
End of Construction, ('Elevated' GWL)	1.1
Long Term following Refuse placement, ('Design' GWL)	1.5

²² Zekkos D et al : 2006 : Unit Weight of Municipal Solid Waste. ASCE Journal of Geotechnical and Geoenvironmental Engineering. Vol 132, No. 10, October 2006, pp 1250 to 1261

Design Scenario	Minimum design FoS (Static) Maximum displacement (Seismic)
Static Loading	
Long Term following Refuse placement, ('Elevated' GWL)	1.1
Seismic Loading	
Long Term following Refuse placement – Design basis earthquake (DBE)	Displacement of liner < 0.3 m Displacement of capping layer <1.0 m
Long Term following Refuse placement – Maximum credible earthquake (MCE)	Displacement of liner < 1.0 m Displacement of capping layer <3.0 m

The above Factors of Safety are considered to adequately reflect both the risk and consequence of failure for a particular loading case, and are compatible with the knowledge gained to date of the performance of the various materials within the landfill development.

9.2.2 Seismic design criteria

Historically, a peak ground acceleration of 0.10 g has been used for the long-term slope stability analyses for design of the Redvale Landfill to date. For the purposes of this Class 1 landfill expansion and Class 2 landfill extension, seismic design criteria have been reassessed, as discussed below.

Peak ground accelerations (PGA) have been calculated for the Design basis earthquake (DBE) and Maximum credible earthquake (MCE), corresponding to 1:475 years and 1: 2,500 years return periods respectively. The PGA values have been calculated using the methodology set out in Module 1 of the NZGS/MBIE guidelines,²³ assuming a Vs30 value of 600 m/s, equivalent to a Class B/C site, as:

- 1 Design basis earthquake (DBE) : PGA = 0.19g
- 2 Maximum credible earthquake (MCE) : PGA = 0.28g

Where a FOS of less than 1.0 is achieved for the seismic loading conditions, the yield acceleration and slope displacement was assessed using 50th percentile values as recommended by the Auckland Council Code of Practice (2023)²⁴. This has been used to estimate the cumulative displacement using the following methods:

- 1 Ambraseys and Srubulov (1995)²⁵
- 2 Jibson (2007)²⁶
- 3 Martin and Qiu (2008)²⁷

The Class 2 landfill extension basegrade cut slopes are likely to have an End-of-Construction design life of only a few years, i.e. before refuse placement to at least the top of the cut basegrade slope. Such assumptions suggest a nominal peak ground acceleration of 0.04 g during the assumed design life of 5 years, with an estimated 1% probability of a DBE peak ground acceleration of 0.19 g.

²³ NZGS and MBIE Module : November 2021 : Earthquake Geotechnical Engineering Practice in New Zealand, Module 1, Rev 1

²⁴ Auckland Council : 2023 : Code of Practice for Land Development and Subdivision

²⁵ Ambraseys N and Srubulov M : 1995 : Earthquake induced displacements of slopes. Soil Dynamics and Earthquake Engineering, 14

²⁶ Jibson R W : 2007 : Regression models for estimating coseismic landslide development. Engineering Geology Vol 91

²⁷ Martin G R and Qiu C : 2008 : In NCHRP Report 611. Transportation Research Board, Washington

9.3 Perimeter bund

9.3.1 General

As noted in Sections 4.1 and 4.2 above, a bund between 2 m and 6 m in height will be constructed along the Eastern and Southern Boundaries, with waste placed up to the crest of those bunds. The alignment of the bunds generally coincides with the existing Landfill perimeter access road, which will either be realigned or constructed on the completed bund.

In order to ensure adequate stability of those bunds under the waste loading, they will need to be constructed of Type 1C structural fill, either founded on competent material or preferably keyed at least 0.5 m into unweathered Northland Allochthon materials (likely mudstone/siltstone). Any low strength and/or compressible Pleistocene Sediments and non-engineered fill below the bunds' alignment should be removed and replaced by Type 1C fill.

The bund profile and geometry will be confirmed as part of detailed design, and how the bund alignment ties into the alignment and profile of the landfill perimeter road. Provisionally, for the purpose of this assessment, the following has been assumed:

- 1 Bund height between 2 m and 6 m above ground level at the existing landfill perimeter (Line A), dependent on geometric requirements.
- 2 Bund fill slopes of 1V:1H and 1V:2H for the inner (adjacent to Line A) and outer fill slopes respectively.
- 3 Depth of any undercut into unweathered Northland Allochthon at least 0.5 m, with undercut slopes formed at no steeper than 1V:1H. Undercut to extend full width of berm and 1 m beyond berm outer edge.
- 4 Any sub-excavation and replacement of Pleistocene Sediments will be carried out carefully (staged) to avoid affecting the existing landfill's integrity. Construction of any such undercut works and placing of engineered fill will follow the requirements of the current Redvale Technical Specification. We also note that the design and construction works will continue to be overseen by the appointed Peer Review Panel, who will review and approve any design and construction methodology in a similar manner as for the landfill construction to date.
- 5 Width of bund upper surface dependent on perimeter road geometry and associated roadside surface water drainage channels. Provisionally expected to be at least 5 m.
- 6 Overall bund width therefore dependent on bund height, (and hence width of fill shoulders), and required access road width and alignment, (if located along bund crest).

9.3.2 Preliminary geotechnical investigations

There is limited historic investigation data outside the landfill perimeter and therefore the distribution, depth and type of Pleistocene Sediments below the proposed bunds' alignment are not known. It is also not known to what extent any existing Pleistocene Sediments were removed in the area adjacent to the landfill perimeter, as part of the earlier construction works. As part of preliminary design, in order to help clarify the depth and nature of the Pleistocene Sediments, and hence the depth and extent of undercut required, a series of shallow investigation boreholes were therefore carried out at nominal 70 m centres along the proposed bund alignment.

The investigations were commissioned by WMNZ and carried out between 28 and 31 October 2025 by Prodrill Ltd under the supervision of Engineering Geology Limited (EGL), who also logged the recovered samples and core. The investigations comprised 15 No boreholes with the upper part "open-barrelled", i.e. no core recovery, and with rotary coring techniques then used to recover core samples of the deeper, more competent materials. The results of the investigations are documented

in EGL's Geotechnical Factual Report²⁸ with the report enclosed as Appendix K. The site plan from EGL's report, showing the borehole locations, is enclosed as Figure B4 in Appendix B and EGL's summary Table 1 of borehole investigations shown below as Table 9.2.

Table 9.2: EGL summary of perimeter bund borehole investigations

Borehole No.	Borehole Depth (m)	Northing	Easting	Ground Level RL (m)	Depth to top of Siltstone from ground surface(m)	Top of Siltstone RL (m)
MBH1	4.5	724316.162	288359.207	63.9	3.2	60.7
MBH2	6	724234.223	288342.184	56	4	52.0
MBH3	3.5	724188.772	288284.951	56.7	2.4	54.3
MBH4	3.5	724128.907	288252.527	56.5	2	54.5
MBH5	3.5	724062.214	288284.36	59.0	2.9	56.1
MBH6	3.5	723975.663	288276.295	58.5	3	55.5
MBH7	1.3	723906.557	288248.122	55.7	0.8	54.9
MBH8	3.5	723843.222	288193.281	51.7	2	49.7
MBH9	2	723771.755	288154.594	51.9	1.3	50.6
MBH10	3.5	723706.341	288124.881	53.9	3.2	50.7
MBH11	7.4	723709.801	288053.689	56.1	6.7	49.4
MBH12	12	723730.21	287977.879	58.4	11.6	46.8
MBH13	3	723738.224	287888.102	60.1	2.7	57.4
MBH14	3.5	723778.542	287807.083	62.3	3.2	59.1
MBH15	7.5	723811.037	287716.874	64.2	3	61.2

9.3.3 Discussion

9.3.3.1 Units encountered

The investigation boreholes described three units, namely:

- 1 Fill: This was encountered as the surface layer in all boreholes, typically comprising hard silty Clay, clayey Silt and dense Gravel. Observed thicknesses varied from 0.3 m to 3.2 m, but typically less than 1.5 m; overlying
- 2 Pleistocene Sediments: This unit was only encountered in Boreholes MBH10 to MBH13, on the eastern half of the southern boundary. Observed thicknesses varied from 1.7 m in MBH10 to a maximum of 8.7 m in MBH12. The materials generally comprised very stiff to hard, brown silty Clay with minor organic debris; overlying
- 3 Northland Allochthon: All boreholes terminated in Northland Allochthon materials generally described as completely weathered grey Siltstone.

Overall the investigations confirm that the ground conditions along the bund alignment were generally as expected from the existing site data and records, with Pleistocene Sediments mantling the Northland Allochthon paleo-topography, but with much of the alluvium having been removed, and replaced by Fill as part of the current landfill development.

²⁸ Engineering Geology Ltd : 21 November 2025 : Redvale Landfill [Perimeter Bund Boreholes] Geotechnical Factual Report. EGL Ref No. 10096

9.3.3.2 Boreholes MBH01 to MBH09

The boreholes along the eastern boundary, i.e. MBH01 to MBH09, identified Fill, from the construction works for the current landfill development and perimeter access roads, directly overlying Northland Allochthon weathered Siltstone, with all Pleistocene Sediments apparently having been removed.

9.3.3.3 Boreholes MBH10 to MBH13

Pleistocene Sediments were encountered below Fill in Boreholes MBH10 to MBH13 on the eastern half of the southern boundary with a maximum of 8.7 m encountered in MBH12. The log of Borehole MBH12 suggests Pleistocene Sediments up to 10.2 m depth, overlying weathered Northland Allochthon, although it is noted the log shows core loss from 6.5 m to 9.0 m. Inspection of the log suggests that Pleistocene Sediments' depth of 10.2 m may be unrealistically deep, bearing in mind the overall geomorphology, the depth of other similar channel features encountered during the current landfill's construction, and construction observations for the current cells along the landfill's southern boundary. Based on that, and our Redvale site experience, it is considered that the lower part of the borehole may instead represent weathered Northland Allochthon mudstone/siltstone.

It is possible that the boreholes encountered a local infilled channel in the underlying Northland Allochthon paleo-topography, and a similar, although shallower feature, was identified in that general area during construction of the southern edge of the Phase 15/16 cells. The extent of that feature, and possible design and construction options for the bund at that location, will need to be confirmed as part of detailed design. If the depth makes full excavation and replacement with engineered fill impractical then other options will need to be considered. These could include only partial excavation/replacement or founding the bund directly over the Pleistocene Sediments bearing in mind the unit's relatively high strength, possibly combined with some form of localised ground improvement such as stone columns or similar.

The Pleistocene Sediments identified along the southern boundary were assessed as generally stiff to very stiff, occasionally hard, implying effective strengths generally similar to those assigned to the Northland Allochthon mudstone and the Type 1C fill. That suggests that if the Pleistocene Sediments are either left in place, and/or replaced at least in part by Type 1C fill, then the assessed slope stability will be generally similar to that currently reported for the Northland Allochthon mudstone, for which a lower-bound strength has been selected. These possible options are assessed and discussed in more detail in Section 9.4.6 below.

It should be noted that the area to the immediate south of the landfilled boundary has been extensively modified during the landfill's development, with construction of ponds and access roads. Some existing sub-excavation and replacement with engineered fill will have occurred as part of those development works.

9.4 Slope stability analysis

9.4.1 General

The slope stability of the proposed Landfill extension works has been analysed using the computer program, SLOPE/W²⁹. The slope stability analyses have been carried out along the following representative cross-sections, using the geological models, groundwater conditions and design parameters assessed in preceding sections of this Report:

- 1 Cross-Sections N1 and N2 through the Northern Boundary:

²⁹ Geo-slope International Ltd : SLOPE/W for stability analysis. Version 4.22.

- Cross-Section N1 is through the western-half of the Northern Boundary, and intersects the stabilising 'Phase 10 Bund' constructed along that portion of the Landfill boundary.
- Cross-Section N2 through the eastern-half of the Northern Boundary.
- 2 Cross-Sections E1, E2 and E3 through the northern and southern parts of the Eastern Boundary respectively.
- 3 Cross-Section S1 through the Southern Boundary.
- 4 Cross-Sections MF1, MF2 and MF3 through the Class 2 extension.

The analyses have considered circular and non-circular (fully specified) surfaces as appropriate, with sensitivity analyses considering various combinations of material strengths and groundwater conditions.

The analyses have been carried out for the following design cases, in accordance with Table 9.1 of the report:

- 1 'Design' groundwater level.
- 2 'High' (Elevated), groundwater level.
- 3 'Design' groundwater levels, with DBE and MCE seismic peak ground accelerations.

9.4.2 Geotechnical model

For the purposes of the stability assessments discussed in this Report, except where specifically discussed, the following assumptions have generally been made with respect to the geotechnical model:

- 1 As discussed in Section 9.2.2, DBE and MCE peak ground accelerations (PGA) of 0.19 g and 0.28 g respectively, have been used for the long-term seismic slope stability analyses for the Redvale Landfill.
- 2 Northern Boundary – Western half:
 - Ground conditions have been based on the as-built records for the Phase 10 Bund.
 - Groundwater conditions are based on data from historic and recent groundwater monitoring.
- 3 Northern Boundary – Eastern Half, Eastern Boundary and Southern Boundary:
 - Ground conditions have been assumed to comprise either Northland Allochthon Unit N7 or Unit N5 mudstones, representing likely lower-bound and upper bound mudstone/siltstone strengths respectively.
 - Groundwater has been assumed to be at either 3 m or 1 m below adjacent existing ground level, corresponding to design or high groundwater conditions respectively.
 - The ground profile beyond the landfill boundary, i.e. 'Line A' has been assumed to be horizontal.
 - A bund, varying in height between 2 m and 6 m above existing ground level will be constructed along the Eastern and Southern Boundaries. The proposed bund construction is discussed further in Section 9.3 above.
- 4 Class 2 Extension:
 - Ground conditions have been based on historic borehole and engineering geology mapping records and data from the recent BH W138.
 - Groundwater has been based on historic data and data from the recent BH W138.
- 5 Waste has been placed at Redvale Landfill since the mid-1990s and it is likely therefore that strengths of that earlier 'mature' waste have increased from those assumed for previous cell

designs and given in Table 8.1. Conservatively, however, those previously assumed strength parameters have been used as the default waste strengths in this assessment, except where specifically noted otherwise.

- 6 The analyses have assumed that any existing final cap will be removed prior to placing the new waste. It has also been assumed that the new waste is mixed with the immediate underlying old (existing) waste to avoid the potential for an interface layer being inadvertently formed.

Reference should also be made to the discussions and clarifications for specific slope stability analysis models in Section 9.4.3 to 9.4.7 below.

9.4.3 'B-bar' stability analyses

If fill is placed rapidly over low strength, compressible soils then excess pore water pressures can be generated which can affect or reduce the assessed slope stability FoS. However the Landfill has been constructed entirely over unweathered Northland Allochthon Mudstone, Siltstone and Limestone, and Waitemata Group Sandstone, with assessed strengths varying from very weak to moderately strong. All soils, such as Pleistocene Sediments, were removed prior to any refuse placement, and a similar approach will be adopted for the proposed extension works.

For the purposes of the Slope/W analyses discussed in this Report, c' and Φ' drained shear strengths have been assumed, effectively treating the founding subgrade as soils although in practice they are unweathered relatively high strength, low compressibility fractured rock.

For waste placed within the landfill, even if it were assumed that placing such waste could potentially induce temporary excess pore water pressures, it is considered that there would be negligible adverse effect on slope stability as the waste is totally enclosed within the landfill void.

The additional waste to be placed on the (inner) edge of the proposed perimeter bunds, to form the revised (steepened) waste profile could, in theory, induce temporary excess pore water pressures, which could in turn affect the slope stability of the landfill/bund interface. Conservatively the potential for such instability has been assessed using the "piezometric Line with B-bar" analysis option within Slope/W.

- a Analyses were carried out along two representative cross-sections E2 and E3, for the elevated groundwater case, and the Slope/W output plots are enclosed in Appendix G.
- b The results of the analyses are summarised below in Table 9.3:

Table 9.3: Slope stability results for B-bar analyses

File name	Surface	Groundwater	FoS		Meets Target FoS?
			No B-Bar	B-Bar	
E2-Elevated -JASL B-Bar	Circular	1 m bgl	2.213	1.177	Yes
E3-Elevated -JASL B-Bar	Circular	1 m bgl	1.926	1.239	Yes

- c The analyses show that, as expected from the analysis assumptions, the assessed FoS decrease for B-bar analysis. However, the assessed FoS are still greater than the target FoS of >1.1 for elevated groundwater conditions.
- d We consider these analyses are conservative for reasons including:
 - i The bunds will be underlain by rock, with any soils having been removed, and that is conservatively modelled in terms of likely pore pressures by the Slope/W analyses;
 - ii The loading on the bunds' inner edge from the landfilling will be gradual as waste is progressively placed up to the final profile, likely over a period of years. If any excess

- pore water pressures were to develop then they will therefore have time to dissipate as the total final loading will not be instantaneous;
- iii As discussed elsewhere in this Report, 'worst-case' conditions have generally been assumed for the slope stability analyses along the selected cross-sections, assuming low strengths for the Northland Allochthon mudstone and maximum elevated groundwater levels. In practice ground and groundwater conditions are expected to be better than those assumed 'worst-case' design parameters and conditions;
 - iv The check B-bar analyses show that FoS still meet target values, even with the conservative and unlikely assumptions regarding generation of excess pore water pressures.
- e In view of the above, the actual slope stability FoS for the refuse/bund interface will be greater than indicated by the B-bar analysis and likely similar to the higher currently reported (non-B-bar) analysis results.

The overall conclusion therefore is that the generation of excess pore water pressures is unlikely, and the current Slope/W model assumptions and analyses are appropriate in view of the conservative assumptions.

9.4.4 Northern boundary

9.4.4.1 General

Pre-landfill development the original topography sloped down towards the north and north-west. As discussed above in Section 3.2, to achieve sufficient airspace volumes a bund (fill embankment) termed the 'Phase 10 Bund' was constructed along the western half of the northern boundary. As a result of instability (translational sliding of fill over weathered Northland Allochthon) a substantial shear key was constructed below the bund, see Figure D2 in Appendix D.

Construction along the eastern half of the northern boundary was more straightforward, with basegrade formed entirely in cut Northland Allochthon materials, see Figure D1 in Appendix D.

9.4.4.2 Cross-section N1

The western one-third of the 'Phase 10 Bund' is underlain by relatively competent limestone (Unit N1) but the remaining eastern portion is underlain by the pervasively sheared Unit N7 mudstone. Cross-section N1, to assess the western half of the Northern Boundary, was therefore positioned to correspond to a location underlain by Unit N7 and through the maximum proposed revised waste height, see Plans and Cross-sections in Appendix B and Appendix C respectively.

The bund was constructed in 1992-1993 and historic slope stability analyses from that period are not readily useable. However, the stability of that portion of the 'Phase 10 Bund' was reassessed as part of the 2014 Phase 19A geotechnical assessment³⁰ and Cross-section 19A-N1 from that report was therefore able to be reused for this study's Cross-section N1. The 2014 SLOPE/W file was updated to the latest SLOPE/W version and has been used for this study.

The analyses have mainly considered the critical failure mode, that is translational movement on the potential shear surfaces inferred to be associated with the sheared mudstone layer. Variables considered have been:

- 1 The assumed strength of that layer as $c' = 0 \text{ kPa}$, $\Phi' = 11^\circ$ or $c' = 0 \text{ kPa}$, $\Phi' = 14^\circ$

³⁰ Tonkin & Taylor Ltd : March 2014 : Transpacific Waste Management. Redvale Landfill Project, Phase 19A Development Geotechnical Assessment Report. Ref 24976.320

- 2 High groundwater at about 1 m below existing groundwater level across the bund, or a design groundwater levels assumed at the top of the mudstone.

The results of selected slope stability analyses along Cross-section N1 are enclosed in Appendix G summarised in Table 9.4.

Table 9.4: Slope stability analysis results for cross-section N1

File name N1-2024-	Surface	Groundwater	Shear surface		FoS	Comment
			c' (kPa)	Phi' (°)		
-A	Circular	High	0	11	1.993	Large radius
-B	Translational	Design	0	11	1.738	---
-C	Translational	High	0	14	1.736	---
-D	Translational	High	0	11	1.325	---
-DBE	Circular	High	0	14	0.987	Seismic Displacement <10 mm
-MCE	Circular	High	0	14	0.777	Seismic Displacement <20 mm

The key conclusions from the stability analyses for Cross-section N1 are:

- 1 For large radius circular slip surfaces, calculated FoS satisfy the design criteria even for high groundwater, a lower assumed shear surface strength and assumed lower strength Unit N7 material.
- 2 For the translational failure mode calculated FoS satisfy the design criteria for design and high groundwater respectively.
- 3 Seismic analysis shows assessed FoS < 1, but with seismic displacements less than 10 mm and 20 mm for the DBE and MCE load cases respectively. The analyses have assumed the previous historic waste strengths but, as discussed in Section 8.2, such waste strengths are likely to be conservative.

Although the stability analyses have generally confirmed that the 'Phase 10 Bund' section of the Northern Boundary will meet stability criteria, the stability analyses, and calculated FoS, are particularly sensitive to assumptions on groundwater. Additional piezometers were installed along that portion of the Northern Boundary in October 2024, (see Section 5.3), to compare with historic groundwater monitoring data. As discussed in Section 7.3.2 above, monitoring of those piezometers confirm that the groundwater conditions assumed in these stability analyses are appropriate.

9.4.4.3 Cross-section N2

The eastern part of the Northern Boundary is formed entirely in cut, predominantly in Northland Allochthon mudstones and siltstones. Cross-section N2 is located towards the eastern end of the boundary to correspond to the greatest depth of cut and highest Option C1 waste profile. Cross-section N2a is located similarly, but with a more oblique alignment, see Plans and Cross-sections in Appendix B and Appendix C respectively.

The analyses have considered large radius circular failure surfaces and variables considered are:

- 1 Northland Allochthon materials have been assumed to be either Unit N7 or Unit N5, and assessed strengths have been assigned as defined in Table 7.1.
- 2 High groundwater and design groundwater levels have been assumed at 1 m and 3 m below the assumed existing ground surface respectively.

The results of selected slope stability analyses along Cross-sections N2 and N2a are enclosed in Appendix G and summarised in Table 9.5.

Table 9.5: Slope stability analysis results for Cross-section N2

File name N2-C1-	Surface	Groundwater	Mudstone/Siltstone		FoS	Comment
			c' (kPa)	Phi' (°)		
-Design GW-N7-APS	Circular	Design	6	22	2.716	Full height
-Design GW-N5-APS	Circular	Design	28	14	2.716	Full height
-Elevated GW-N7-APS	Circular	High	6	22	3.918	Part height
-Elevated GW-N5-APS	Circular	Design	6	22	4.499	Full height
-APS seismic DBE	Circular	Design	6	22	1.337	Full height
-APS seismic MCE	Circular	Design	6	22	1.028	Full height

The key conclusion from the stability analyses for Cross-section N2 is:

- 1 For large radius circular slip surfaces, calculated FoS satisfy the design criteria for design groundwater, high groundwater and seismic loads respectively, even assuming lower strength Unit N7 material.

9.4.5 Eastern boundary

9.4.5.1 General

A perimeter bund, generally as discussed in Section 9.3 above, is proposed to be constructed along the Eastern Boundary with the new waste sloping up from the top edge of the bund. The proposed bund height will vary between about 2 m and 6 m, depending on the existing ground level along the landfill boundary. Ground conditions along the Eastern Boundary are assessed to be mixture of Northland Allochthon mudstone and siltstone units. For the purposes of this stability assessment the analyses have conservatively assumed:

- 1 Large radius circular failure surfaces.
- 2 Ground conditions to comprise the lower strength Northland Allochthon Unit N7 material with strengths assigned as defined in Table 8.1.
- 3 Elevated groundwater at 1 m below the assumed existing ground surface.

9.4.5.2 Cross-section E1

The results of selected slope stability analyses along Cross-section E1, (see Plans and Cross-sections in Appendix B and Appendix C respectively), with a perimeter berm height of 2 m, are enclosed in Appendix G summarised in Table 9.6.

Table 9.6: Slope stability analysis results for Cross-section E1

File name E1-	Surface	Groundwater	Mudstone/Siltstone		FoS	Comment
			c' (kPa)	Phi' (°)		
-Elevated-N7-APS	Circular	1 m bgl	6	22	2.512	Full height
-Elevated-N7-APS-seismic DBE	Circular	1 m bgl	6	22	1.167	Seismic

File name E1-	Surface	Groundwater	Mudstone/Siltstone		FoS	Comment
			c' (kPa)	Phi' (°)		
-Elevated-N7-APS-seismic MCE	Circular	1 m bgl	6	22	0.895	Seismic Displacement <10 mm

The key conclusion from the stability analyses for Cross-section E1 is:

- 1 For large radius circular slip surfaces, calculated FoS satisfy the design criteria for design groundwater, high groundwater and seismic loads respectively, even assuming lower strength Unit N7 material.

9.4.5.3 Cross-section E2

The results of selected slope stability analyses along Cross-section E2, with a perimeter berm height of 2 m, are enclosed in Appendix G and summarised in Table 9.7.

Table 9.7: Slope stability analysis results for Cross-section E2

File name E2-	Surface	Groundwater	Mudstone/Siltstone		FoS	Comment
			c' (kPa)	Phi' (°)		
-Elevated-APS	Circular	1 m bgl	6	22	2.213	Full height
-Elevated- APS-seismic DBE	Circular	1 m bgl	6	22	1.097	Seismic
-Elevated- APS-seismic MCE	Circular	1 m bgl	6	22	0.864	Seismic Displacement <10 mm

The key conclusion from the stability analyses for Cross-section E2 is:

- 1 For large radius circular slip surfaces, calculated FoS satisfy the design criteria for design groundwater, high groundwater and seismic loads respectively, even assuming lower strength Unit N7 material.

9.4.5.4 Cross-section E3

The results of selected slope stability analyses along Cross-section E3, with a perimeter berm height of 6 m, are enclosed in Appendix G and summarised in Table 9.8.

Table 9.8: Slope stability analysis results for Cross-section E3

File name E3-	Surface	Groundwater	Mudstone/Siltstone		FoS	Comment
			c' (kPa)	Phi' (°)		
-Elevated-APS	Circular	1 m bgl	6	22	1.926	Full height
-Elevated-APS-seismic DBE	Circular	1 m bgl	6	22	1.037	Seismic
-Elevated-APS-seismic MCE	Circular	1 m bgl	6	22	0.856	Seismic Displacement <10 mm

The key conclusion from the stability analyses for Cross-section E3 is:

- 1 For large radius circular slip surfaces, calculated FoS satisfy the design criteria for design groundwater, high groundwater and seismic loads respectively, even assuming lower strength Unit N7 material.

9.4.6 Southern boundary

9.4.6.1 General

In a similar manner to the Eastern Boundary, a 2 m high perimeter bund, will be constructed along the Southern Boundary. The perimeter bund is proposed to extend for about 400 m westwards along the boundary to opposite (north of) the current workshop area. Ground conditions along the Southern Boundary are assessed to be mixture of Northland Allochthon mudstone and siltstone units. For the purposes of this concept stability assessment the analyses have conservatively assumed:

- 1 Large radius circular failure surfaces.
- 2 Ground conditions to comprise the lower strength Northland Allochthon Unit N7 material with strengths assigned as defined in Table 7.1.
- 3 Elevated groundwater at 1 m below the assumed existing ground surface.

9.4.6.2 Cross-section S1

The results of selected slope stability analyses along Cross-section S1, (see Plans and Cross-sections in Appendix B and Appendix C respectively), are enclosed in Appendix G and summarised in Table 9.9.

Table 9.9: Slope stability analysis results for Cross-section S1

File name S1-	Surface	Groundwater	Mudstone/Siltstone		FoS	Comment
			c' (kPa)	Phi' (°)		
-Elevated-APS	Circular	1 m bgl	6	22	1.964	Full height
-Elevated- APS- seismic DBE	Circular	1 m bgl	6	22	1.003	Seismic
-Elevated- APS- seismic MCE	Circular	1 m bgl	6	22	0.797	Seismic Displacement <20 mm

The key conclusion from the stability analyses for Cross-section S1 is:

- 1 For large radius circular slip surfaces, calculated FoS satisfy the design criteria for design groundwater, high groundwater and seismic loads respectively, even assuming lower strength Unit N7 material.

9.4.6.3 Pleistocene Sediments identified in Boreholes MBH10 to MBH13

As discussed in Section 9.3 above, Pleistocene Sediments were encountered below Fill in Boreholes MBH10 to MBH13 on the eastern half of the southern boundary. A maximum of 8.7 m was encountered in MBH12, with the interface with underlying weathered Northland Allochthon at 10.2 m depth. The perimeter bund along that portion of the southern boundary is adjacent to Cross-section S1, and stability analyses have been carried out using that cross-section to provide an initial geotechnical assessment of possible undercut excavation and replacement options for construction of the perimeter bund. The possible options, as noted in Section 9.3.3, and to be confirmed as part of detailed design, are:

- a Option 1: Assuming a 10 m depth of Pleistocene Sediments, with the upper 5 m replaced by Type 1C engineered fill. Such a replacement depth better matches the assessed channel depth as discussed above;
- b Option 2: Assuming the full 10 m thickness of Pleistocene Sediments, as identified by Borehole MBH12, is replaced by Type 1C engineered fill;
- c Option 3: Assuming the assessed 10 m depth of Pleistocene Sediments is left in place. However, being alluvial deposits, spatial variation in composition and strength can be expected, and Options 1 and 2 discussed above are considered to give increased confidence in terms of subgrade quality. If this option is considered as part of detailed design then it will likely require further investigations to confirm consistent ground conditions below the proposed bund alignment

Stability analyses have been carried out to assess the practicality of the three options and the Slope/W stability plots are enclosed in Appendix G and summarised in Table 9.10:

Table 9.10: Slope stability analysis results for undercut and replacement options

File name S1-Elevated-APS-	Surface	Groundwater	Mudstone/Siltstone		FoS	
			c' (kPa)	Phi' (°)	Full height	Minimum
-Case B (Option 1)	Circular	Design	6	22	2.064	1.812
-Case A (Option 2)	Circular	Design	6	22	2.064	1.812
-Case C (Option 3)	Circular	Design	6	22	2.043	1.812

Note that for the purposes of this conceptual option assessment, the Pleistocene Sediments and Type 1C fill have been modelled as continuous layers. That is a conservative assumption as the Pleistocene Sediments would have been removed from immediately adjacent to the Landfill cell highwall prior to liner placement and any replacement of Type 1C fill would only extend a finite distance from the southern edge of the perimeter bund.

The stability analyses indicate that assessed FoS for the possible construction options comfortably meet the target FoS and are similar to the FoS for the general Cross-section S1 stability assessment discussed in Section 9.4.6.2. The detailed design will be dependent on the results of construction excavation observations, and possibly additional investigations, to confirm the extent and nature of the infilled channel. However, other similar features encountered during the current landfill's construction suggest the channels are steep sided, and of relatively limited extent.

9.4.7 Class 2 extension

9.4.7.1 General

As discussed in Section 4.1 it is proposed to extend the landfill profile as a 'lobe' for placement of Class 2 waste, constructed southwards and south-eastwards over the former Managed Fill area.

Ground conditions below the proposed Class 2 waste extension area are assessed as historic Fill, overlying Pleistocene Sediments, with the underlying bedrock represented by Northland Allochthon Mudstone, see Section 7.2.4 above. The Thrust Fault, separating Northland Allochthon and Waitemata Group materials, is inferred to be towards the western boundary of the proposed Class 2 landfill extension area.

For the purposes of this stability assessment, Cross-sections MF1, MF2 and MF3, (see Plans and Cross-sections in Appendix B and Appendix C respectively), have been developed through the extension area and have conservatively assumed the following:

- 1 Stability analyses have been carried out through the cut basegrade profile, i.e. for the temporary (end-of-construction) case prior to waste placement, for all three cross-sections. Analysis of the long-term case, i.e. following waste placement, has been carried out for Cross-section MF3 which is oriented along the full length/height of the proposed cell.
- 2 Large radius circular failure surfaces.
- 3 The Northland Allochthon bedrock has been assumed to comprise the lower strength Unit N7 material, with strengths assigned as defined in Table 8.1.
- 4 The groundwater profile, for the design and elevated groundwater cases, have been selected based on the discussions in Section 7.3.3 above.
- 5 The shear keys constructed for Sedimentation Pond 6 and the North and South Haul Roads have conservatively not been modelled for this concept analysis.
- 6 WMNZ has advised that soils placed in the Managed Fill have now been substantially removed and the analysis has assumed that basegrade will therefore comprise either unweathered Northland Allochthon materials, or Type 1C structural fill if local undercutting/filling is required.

9.4.7.2 Cross-section MF1

The results of selected slope stability analyses along Cross-section MF1, oriented east-west through the cell's western highwall, are enclosed in Appendix G and summarised in Table 9.11.

Table 9.11: Slope stability analysis results for Cross-section MF1

End-of-construction case (no waste placed)						
File name MF1-	Surface	Groundwater	Mudstone/Siltstone		FoS	Comment
			c' (kPa)	Phi' (°)		
-Baseline-APS	Circular	Design	6	22	1.286	Full height
-Elevated-APS	Circular	Elevated	6	22	1.102	Full height

The key conclusion from the stability analyses for Cross-section MF1 is:

- 1 For large radius circular slip surfaces, calculated FoS satisfy the design criteria for design groundwater, high groundwater and seismic loads respectively, even assuming lower strength Unit N7 material.

9.4.7.3 Cross-section MF2

The results of selected slope stability analyses along Cross-section MF2, oriented SW - NE through the cell's south-western highwall, are enclosed in Appendix G and summarised in Table 9.12.

Table 9.12: Slope stability analysis results for Cross-section MF2

End-of-construction case (no waste placed)						
File name MF2-	Surface	Groundwater	Mudstone/Siltstone		FoS	Comment
			c' (kPa)	Phi' (°)		
-Elevated-N7-APS	Circular	Elevated	6	22	1.443	Full height

The key conclusion from the stability analyses for Cross-section MF2 is:

- 1 For large radius circular slip surfaces, calculated FoS satisfy the design criteria for design groundwater, high groundwater and seismic loads respectively, even assuming lower strength Unit N7 material.

9.4.7.4 Cross-section MF3

The results of selected slope stability analyses along Cross-section MF3, oriented NW - SE through the cell's full length/height, are enclosed in Appendix G and summarised in Table 9.13.

Table 9.13: Slope stability analysis results for Cross-section MF3

End-of-construction case (no waste placed)						
File name MF3-	Surface	Groundwater	Mudstone/Siltstone		FoS	Comment
			c' (kPa)	Phi' (°)		
-Basegrade-Design-APS	Circular	Design	6	22	1.311	Full height
Long-term case (waste placed)						
-Waste-Elevated-APS	Circular	1 m bgl	6	22	2.115	Full height
-APS New Waste DBE	Circular	1 m bgl	6	22	1.131	New waste strength Seismic
-APS New Waste MCE	Circular	1 m bgl	6	22	0.900	New waste strength Seismic Displacement <20 mm

The key conclusion from the stability analyses for Cross-section MF3 is:

- 1 For large radius circular slip surfaces, calculated FoS satisfy the design criteria for design groundwater, high groundwater and seismic loads respectively, even assuming lower strength Unit N7 material.

9.4.8 Discussion

Slope stability analyses have been carried out along 10 No. representative cross-sections through the proposed development. The results of the analyses show that:

- 1 Northern Boundary, Eastern Boundary and Southern Boundary:
 - The assessed stability of the proposed development final waste slope profile satisfies design criteria for design groundwater, elevated (high) groundwater and seismic loads.
- 2 Class 2 waste extension:
 - The proposed Class 2 extension basegrade slopes satisfy design criteria for design groundwater, elevated (high) groundwater and seismic loads.
 - The assessed stability of the proposed Class 2 extension waste slope profile satisfies design criteria for design groundwater, elevated (high) groundwater and seismic loads.

The above conclusions are based on stability analyses carried out assuming likely 'worst-case' material strength parameters and groundwater conditions. In practice material strengths are likely to be greater than those assumed 'worst-case' values, over some or all of the development area. Stability FoS will therefore likely further increase beyond the assessed, already acceptable, values.

9.5 Earthworks and construction issues

9.5.1 Technical specification

An initial Technical Specification was prepared in October 1993 and updated, with minor amendments to incorporate design and construction lessons learnt to date, in April 2015³¹. The April 2015 Specification has subsequently been used for all landfill construction, and will continue to be used for the proposed Landfill extension works.

The Technical Specification documents the key design and construction philosophy, in particular defined Fill Types, characterisation of the main basegrade geological units, Quality Control (QC) requirements, and general earthworks requirements. Section C3.0: Earthworks, from the April 2001 Technical Specification document, is enclosed as Appendix I. Considering the key aspects in turn:

- 1 Section 2 of the Technical Specification defines 12 No. Fill Types, corresponding to the material properties and descriptions appropriate to their proposed end use, e.g. generally either for structural fill or for low-permeability liner.
- 2 Quality Control testing requirements are specified in Section 2.14 of the Technical Specification.
- 3 The principal geological units expected to be encountered at the site have been identified in Section 2.3 of the Technical Specification.
- 4 The remaining sections of the Section 2 earthworks portion of the Technical Specification relate to general earthworks cut and fill methodology and requirements.

9.5.2 General construction procedures

Earthwork construction will be carried out as per the requirements of the Technical Specification,³² discussed above.

The general approach is that selected materials, obtained either directly from on-site borrow areas or from stockpiles, have been placed in thin loose layers. They have then either been spread by 'dozers and then compacted with rollers, or spread and compacted using 4 x 4 sheepfoot rollers with blade attachments, or other dedicated compaction plant.

Within landfill cells, prior to any fill placement the cut subgrade has been inspected by the Liner Review Team, (see Section 9.5.4) and any necessary further undercut, trimming or sub-soil drainage works carried out. A similar process has also carried out for other areas to be backfilled, such as shear key bases or fill undercut areas.

Similar earthworks and construction procedures to those used to date for the Redvale Landfill will apply to the proposed extension works, in particular to form the basegrade excavation and liner works below the proposed Class 2 waste extension area to the south of the current Landfill boundary.

9.5.3 Structural and bulk fill types

Twelve Fill Types are defined in the Technical Specification, (see Appendix I), of which Types 1A, and 1C are the most commonly used structural fill types, and Types 3A, 3B and 5A the most commonly used bulk fill types. All such fills are formed from compacted Northland Allochthon materials, either Mudstone/Siltstone or Limestone depending on the fill type, and excavated from on-site borrow areas.

³¹ Tonkin & Taylor Ltd : March 2015 : Specification. Redvale Landfill General Construction. Ref 30089.

³² Tonkin & Taylor Ltd : March 2015 : Specification. Redvale Landfill General Construction. Ref 30089.

To form the proposed final waste profile along the eastern and southern sides of the Landfill boundary bunds up to 6 m high will be required, see Section 9.3 above. Those bunds and any undercut or shear key excavation will be backfilled with Type 1C fill.

9.5.4 Liner fill

The Technical Specification,³³ defines Types 1A and 1B fills as suitable for construction of the low-permeability lining system and requires the measured coefficient of permeability of placed liner to be less than 1×10^{-9} m/sec. In practice all lining systems installed at the site to date are understood to have been constructed using Type 1A Fill, corresponding to selected Mudstone or Siltstone, and virtually all tests over the past 25 years have shown that permeability value to be readily achievable. It is understood the lining system for the Class 2 extension will only require a coefficient of permeability of less than 1×10^{-8} m/sec, which it is therefore considered will be readily achievable for Type 1A fill, based on the historic testing.

9.5.5 Quality assurance and quality control testing

The Quality Control (QC) testing requirements and criteria are set out in the Technical Specification,³⁴ in Section C3.0 : Earthworks enclosed in Appendix I. To date all Liner QC testing, and the majority of structural fill testing, has been carried out by the independent testing contractor Geotechnics Ltd. The same testing regime, with strict compliance to the Technical Specification, will also apply to any fill placement for the proposed Class 2 extension works.

The Quality Assurance, (QA) processes carried out at Redvale Landfill comprise a number of components, namely carrying out regular site visits by T+T geotechnical staff. Such visits are in part to carry out the Basegrade Inspection mapping required by the consents, and provide general geotechnical advice to WMNZ on ongoing construction works. Similar QA procedures will apply for the proposed Class 2 waste extension works with, in particular, mapping of any new basegrade, or exposed areas of historic basegrade, by the Liner Review Team, (see Section 9.5.4 above).

9.5.6 Construction risk

Provided the design and construction recommendations given in this Report are complied with, it is considered that the risk of adverse ground movements during construction can be reduced to an acceptable level. However, site experience to date has shown that the geological structure is complex, with low angle shear zones and other discontinuities within the rock mass. The distribution of such features is difficult to identify from surface mapping and boreholes and the potential for unknown adverse features or other unfavourable ground conditions to be present cannot be discounted.

Therefore, to minimise the potential for adverse ground movements potentially affecting the landfill construction, during the excavation, and formation of the design slopes, the works should be regularly inspected and mapped by an experienced Engineering Geologist and/or Geotechnical Engineer to:

- 1 Confirm that ground and groundwater conditions as encountered are in general agreement with those assumed for design.
- 2 Allow any potentially unfavourable shear zones or other discontinuities to be identified such that design and construction procedures can be modified if necessary.
- 3 Allow potential sources of liner and other fill materials to be confirmed.

³³ Tonkin & Taylor Ltd : March 2015 : Specification. Redvale Landfill General Construction. Ref 30089.

³⁴ Tonkin & Taylor Ltd : March 2015 : Specification. Redvale Landfill General Construction. Ref 30089.

Such input forms the basis of the Quality Assurance and Quality Control Testing inputs discussed above.

In addition, the cell construction works should generally be programmed to minimise the length of time before refuse is placed against the unsupported toe of any cut slopes.

9.6 Geotechnical monitoring

Geotechnical monitoring has formed an integral part of construction works at Redvale Landfill to date and should continue to do so during the construction of the proposed Landfill extension works. Such monitoring generally comprises assessment of groundwater levels and pressures, and ground movements of cut and fill slopes. The typical approach to be adopted is to define acceptable design groundwater levels or ground displacements and then monitor such parameters during construction. Should the monitoring identify groundwater levels or ground displacements exceeding design values, i.e. “trigger” levels, then pre-determined contingency measures may need to be implemented.

The investigations, design assessments and construction observations at the Redvale Landfill to date have shown that slope stability is sensitive to assumptions regarding groundwater conditions. This is also applicable to the currently proposed landfill extension works, with the slope stability analyses discussed in Section 9.4 above showing assessed slope stability to be sensitive to assume current and possible future groundwater levels. Groundwater monitoring will continue during and after construction, in similar manner and frequency as currently, to confirm that groundwater levels and piezometric pressures meet the criteria and values assumed in design.

If during construction, it is considered that additional groundwater information is required then piezometers should be installed to allow design groundwater conditions to be assessed with confidence. Such piezometers would likely be similar to those installed recently (October 2024), i.e. low-volume piezometers grouted into cored or non-core boreholes.

Groundwater monitoring during construction, should typically comprise:

- 1 An ongoing review of existing piezometer data and comparison of the monitored levels against the groundwater levels, or piezometric pressures, assumed for design.
- 2 Recording and monitoring any groundwater seepages observed during construction.

If piezometric levels rise above pre-determined “trigger” levels as identified by the stability analyses, or groundwater seepage rates cause concern, then contingency measures may need to be implemented. Typically, these could include increasing the frequency of groundwater monitoring and observations, modifying the construction procedures and/or programme, or installing drainage works.

10 Site suitability

Studies to assess the potential suitability of the current Redvale Landfill site for use as a sanitary landfill were commenced by WMNZ in 1988. A series of assessments, investigations and design studies were carried out and, following a hearings process, consents for the development of the Redvale Landfill in its current form were granted in 1992, with the consents extended in 2014.

The current resource consents were granted based on ongoing investigations and design studies, with to date those works including:

- 1 Geotechnical investigations comprising boreholes, test pits, large diameter shafts, Cone Penetrometer Tests, seismic refraction surveys and extensive engineering geology mapping.
- 2 Installation and regular monitoring of piezometers, borehole inclinometers and survey pins.
- 3 In-situ and laboratory testing, particularly focusing on material strengths and permeabilities.
- 4 Observations during construction of the, in particular ground conditions, structural fabric, and groundwater behaviour including seepages both from excavations and also from sub-soil drainage works.
- 5 Extensive Quality Control testing of site-won materials to assess their suitability for both structural fill and for the low-permeability liner required by the consents.

The above information has allowed comprehensive, detailed and robust geotechnical, geological and groundwater models to be developed for the site. Those models in turn have enabled the design process to develop appropriate cut and fills slopes, and a low-permeability liner, with confidence, and which are able to be successfully constructed so as to meet the agreed and consented design and performance criteria.

The geotechnical investigation, monitoring, design and construction procedures and processes developed since 1991 have allowed the successful, safe and consent-compliant development of the existing completed landfill cells.

The overall conclusions of this Geotechnical Assessment Report are that:

- 1 From a geotechnical perspective, it is considered that the procedures and processes to date have been appropriate to ensuring that the landfill has been constructed in compliance with the consent and design criteria and requirements.
- 2 It is intended that the geotechnical procedures and processes successfully adopted and followed to date will continue to be used for the design and construction of the proposed extension works, modified as necessary to allow for local variations in geotechnical conditions, landfill cell layout and any other specific design or construction requirements.

On that basis we assess that the site is therefore suitable for the construction of the proposed landfill extension works, comprising steepening the current final landfill profile by means of a waste overlay and extending the landfill's area to the south.

11 Conclusions

Based on the available information the following summarises our conclusions with respect to the geotechnical design and construction of the proposed extensions to the Redvale Landfill:

- 1 A number of geotechnical and hydrogeological studies were carried out for the Redvale Landfill prior to 1992 to support the original resource consent application. Since that date a large number of geotechnical and related studies and reports have been prepared by T+T as the landfill has been progressively developed. Most recently 3 No. additional machine boreholes were carried out along the Landfill's northern boundary and 1 No. machine borehole within the proposed extension area to the south-west of the landfill, with piezometers installed in each borehole.
- 2 An extensive programme of geotechnical instrumentation has also been progressively installed since 1988 at the Redvale Landfill site, principally comprising piezometers, inclinometers and survey pins.
- 3 The results of the studies and investigations have been used to develop an Engineering Geology model for the Redvale Landfill site, which has been progressively revised and upgraded as the landfill development proceeded. The model shows the site to be underlain by approximately north-south trending (Northland Allochthon) Limestone bands, separated by Mudstones and Siltstones, and with (Waitemata Group) Sandstones and Conglomerates present across the south-west corner of the site and to the south and west of the landfill boundary.
- 4 The Eastern Property and the eastern two-thirds of the Western Property are underlain by sheared, generally NNE to SSW trending Northland Allochthon deposits. These are dominated by a prominent central (Mahurangi) Limestone mass aligned approximately along the Properties' common boundary, flanked to the west and east by sheared, often contorted, interbanded Mudstones and Siltstones.
- 5 Waitemata Group Sandstones and Siltstones form the bedrock geology to the south and west of the Redvale Landfill boundary. The inferred junction between those units and the Northland Allochthon materials is delineated by a steeply WSW dipping thrust fault, which runs approximately NNW to SSE across the south-western edge of the Western Property and extends southwards across the western edge of the proposed Class 2 extension. Along the thrust fault is a zone of Autoclastic Breccia, formed during the Allochthon emplacement process, with further, irregularly interbanded, sub-parallel bands of Limestone, Autoclastic Breccia and Waitemata Group extending south-westwards towards and beyond the Landfill boundary and the Waitemata Group country rock.
- 6 Prior to development and basegrade excavation, Pleistocene Sediments and/or Northland Allochthon residual soils formed the immediate surface materials over much of the study area. Low angle-shear planes are often present at the base of those Pleistocene Sediments and within the underlying weathered Northland Allochthon mudstones and siltstones.
- 7 Variable thicknesses and types, of engineered fill have also been placed over the parts of the development area since 1988 and further fill will be necessary to complete the works.
- 8 The hydrogeological regime and groundwater model for the overall Redvale Landfill site is discussed in the Hydrogeology Report, which should be referred to for further detail.
- 9 Geotechnical design parameters have been assigned to the various geological units and fill materials based on geotechnical assessments and investigations to date.
- 10 All cut slopes around the perimeter of the current Landfill void (prior to waste placement) have been formed at 1V:3H, varying to about 1V:4H depending on the slopes' height and ground and groundwater conditions. The great majority of the basegrade cut slopes were in in-situ Northland Allochthon, with the exception of the upper portions of the highwall in the

south-west corner of the Landfill which was formed in Waitemata Group materials. Fill slopes have also been formed at 1V:3H or flatter.

- 11 Pre-landfill development the original topography sloped down towards the north and north-west. To achieve sufficient airspace volumes a bund (fill embankment) termed the 'Phase 10 Bund' was constructed along the western half of the northern boundary.
- 12 The proposed works to create that additional airspace comprise two principal construction components, namely:
 - The current final design landfill ground profile varies but is everywhere less than 1V:3H. That ground profile will be steepened to a maximum grade of 1V:3H to allow placement of additional waste. Along the Eastern and Southern Boundaries bunds, varying from 2 m to 6 m high, will be constructed along the current landfill perimeter to allow the additional waste volumes to be achieved.
 - The landfill will be extended to the south-west of the current Landfill boundary, below the area of the now decommissioned Managed Fill and to the east of Sedimentation Pond 6. The extension will comprise the excavation and lining of a new landfill cell, to allow placement of Class 2 waste, and constructed in a similar manner to the earlier cells within the current landfill.
- 13 The design criteria for the stability of the proposed cut and fill for the Redvale Landfill have been based on the conditions proposed for the Auckland Regional Landfill (ARL) development. Design basis earthquake (DBE) and Maximum credible earthquake (MCE) peak ground accelerations of 0.19 g and 0.28 g respectively have been used for the long-term slope stability seismic analyses for the Redvale Landfill. The Class 2 extension development "temporary" slopes are likely to have a design life of between about 1 and 6 years, hence lower pga values have been used.
- 14 Slope stability analyses have been carried out along 10 No representative cross-sections through the proposed development. The results of the analyses show that:
 - Northern Boundary, Eastern Boundary and Southern Boundary:
 - o The assessed stability of the proposed development final waste slope profile satisfies design criteria for design groundwater, elevated (high) groundwater and seismic loads.
 - Class 2 waste extension:
 - o The proposed Class 2 extension basegrade slopes satisfy design criteria for design groundwater, elevated (high) groundwater and seismic loads.
 - o The assessed stability of the proposed Class 2 extension waste slope profile satisfies design criteria for design groundwater, elevated (high) groundwater and seismic loads.

The above conclusions are based on stability analyses carried out assuming likely 'worst-case' material strength parameters and groundwater conditions. In practice material strengths could be greater than those assumed 'worst-case' values, over some or all of the development area. Stability FoS will therefore likely further increase beyond the assessed, already acceptable, values. Stability analyses using the 'piezometric line with B-bar' approach were also carried out which concluded that generation of excess pore water pressures, as a result of the proposed extension works, is unlikely.

- 15 Investigations for the proposed perimeter bund along the Southern Boundary have identified Pleistocene Sediments locally up to 10.2 m depth, likely representing an infilled channel in the underlying Northland Allochthon paleo-topography. The extent of that feature, and possible design and construction options for the bund at that location, will need to be confirmed as part of detailed design. Provisionally options could include excavation and replacement with

- engineered fill, partial excavation/replacement, or founding the bund directly over the Pleistocene Sediments.
- 16 The proposed development works will be constructed to the requirements of the Technical Specification used for the current Landfill development, which documents the key design and construction requirements.
 - 17 The liner material will be selected Northland Allochthon Mudstone or Siltstone, as used successfully for all existing low-permeability liner at the Redvale Landfill. The material will be sourced either from excavations or from selected stockpile borrow areas. Based on the results of extensive permeability testing to date it is expected that such proposed liner materials will readily be able to meet the required coefficient of permeability for the Class 2 extension liner of less than 1×10^{-8} m/sec.
 - 18 During construction the works should be regularly inspected and mapped by an experienced Engineering Geologist and/or Geotechnical Engineer, to confirm agreement with the design assumptions.
 - 19 On the basis of the reviews and analyses set out in this Geotechnical Assessment Report we consider that, from a geotechnical perspective, the site is therefore suitable for the construction of the proposed landfill extension works, comprising steepening the current final landfill profile by means of a waste overlay and extending the landfill's area to the south.

12 Applicability

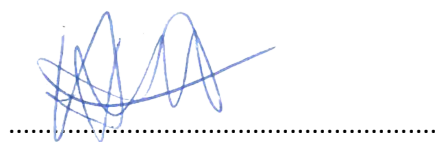
This report has been prepared for the exclusive use of our client WM New Zealand, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

We understand and agree that our client will submit this report as part of an application under the Fast-track Approvals Act 2024 and that an Expert Panel as the consenting authority will use this report for the purpose of assessing that application. We understand and agree that this report will be used by the Expert Panel in undertaking its regulatory functions.

Tonkin & Taylor Ltd
Environmental and Engineering Consultants

Lead author:

Report reviewed by:



Andrew Stiles
Senior Principal Geotechnical Engineer



Tim Coote
Senior Principal Engineering Geologist

Authorised for Tonkin & Taylor Ltd by:



Simonne Eldridge
Project Director

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Appendix A Code of Conduct

Compliance with the Environment Court Practice Note 2023

I confirm that, in my capacity as lead author of this report, I have read and abided by the Environment Court of New Zealand's Code of Conduct for Expert Witnesses contained in the Practice Note 2023.

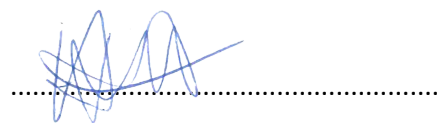
I am a Senior Principal Geotechnical Engineer at Tonkin & Taylor Ltd (T+T), where I specialise in Geotechnical Engineering and Engineering Geology. I have worked at T+T since 1995, initially in Malaysia, before relocating to Auckland in 2001. Prior to joining T+T, I was the Manager of the Geotechnical Group in Works Consultancy Services' Wellington office.

I have 50 years' experience in Geotechnical Engineering and Engineering Geology. I am a Chartered Member of Engineering New Zealand (CMEngNZ) and a Chartered Professional Engineer (CPEng). I hold the following qualifications:

- MSc, Imperial College, London University
- BSc, Imperial College, London University
- ARSM, DIC

Recent relevant projects and services that I have been involved with include:

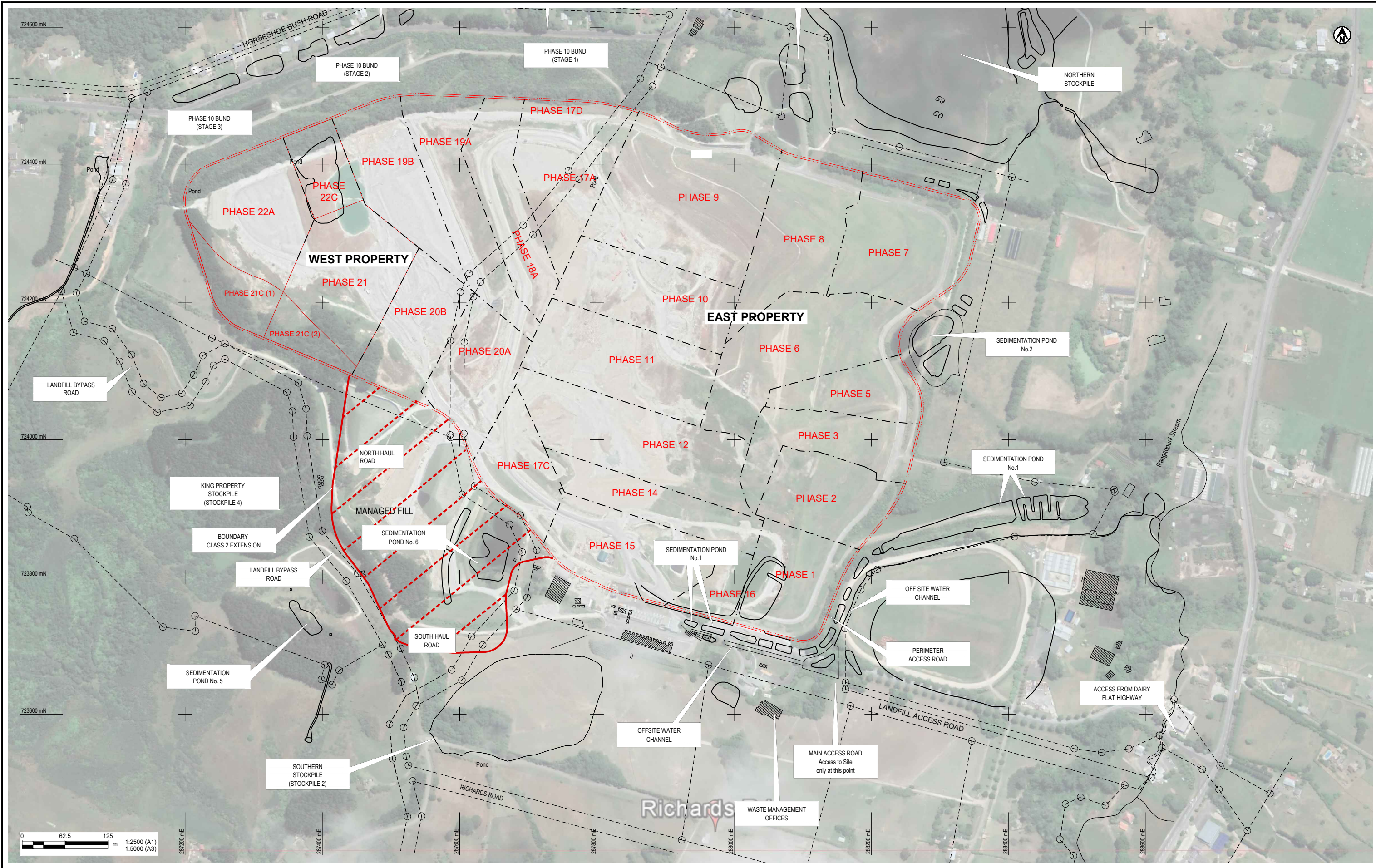
- **Redvale Landfill, Auckland** – Responsible for all geotechnical and engineering geology design and construction aspects for the Redvale Landfill, the largest waste disposal site in New Zealand.
- **Whitford Landfill, Auckland** – Responsible for all geotechnical design and construction aspects for Whitford Landfill, including cell construction and the design, excavation and stabilisation of multi-bench slopes in fractured greywacke within the former quarry.
- **Tirohia Landfill, Waikato** – Stability reviews of the existing Tirohia Landfill and technical review of the geotechnical aspects of the design of the proposed replacement Tirohia Landfill
- **Millwater Development, Auckland** – Geotechnical management and technical overview of major sub-division development, with substantial earthworks including geotechnical investigation, design and construction overview of MSE slopes, walls and slope stabilisation works.
- **Advice to Auckland Council** – Provision of emergency response advice to Auckland Council for natural hazard events and construction incidents, including providing Expert Witness evidence in the Environment Court.
- **Clevedon - Kawakawa Road, Auckland** – Substantial illegal filling works have been carried out in a marine intertidal zone. Carried out a detailed site inspection and assessment followed by preparation of an expert report for Council.
- **Wiri North Quarry, Manukau** – Involved in the progressive decommissioning and backfilling of the Wiri North Quarry.
- **Waterview SH20 Connection, Auckland** – Advised Auckland Council on potential effects of the Waterview SH20 Connection tunnels on overlying, previously landfilled Council owned Reserves. Gave Expert Witness Evidence at the Board of Enquiry.



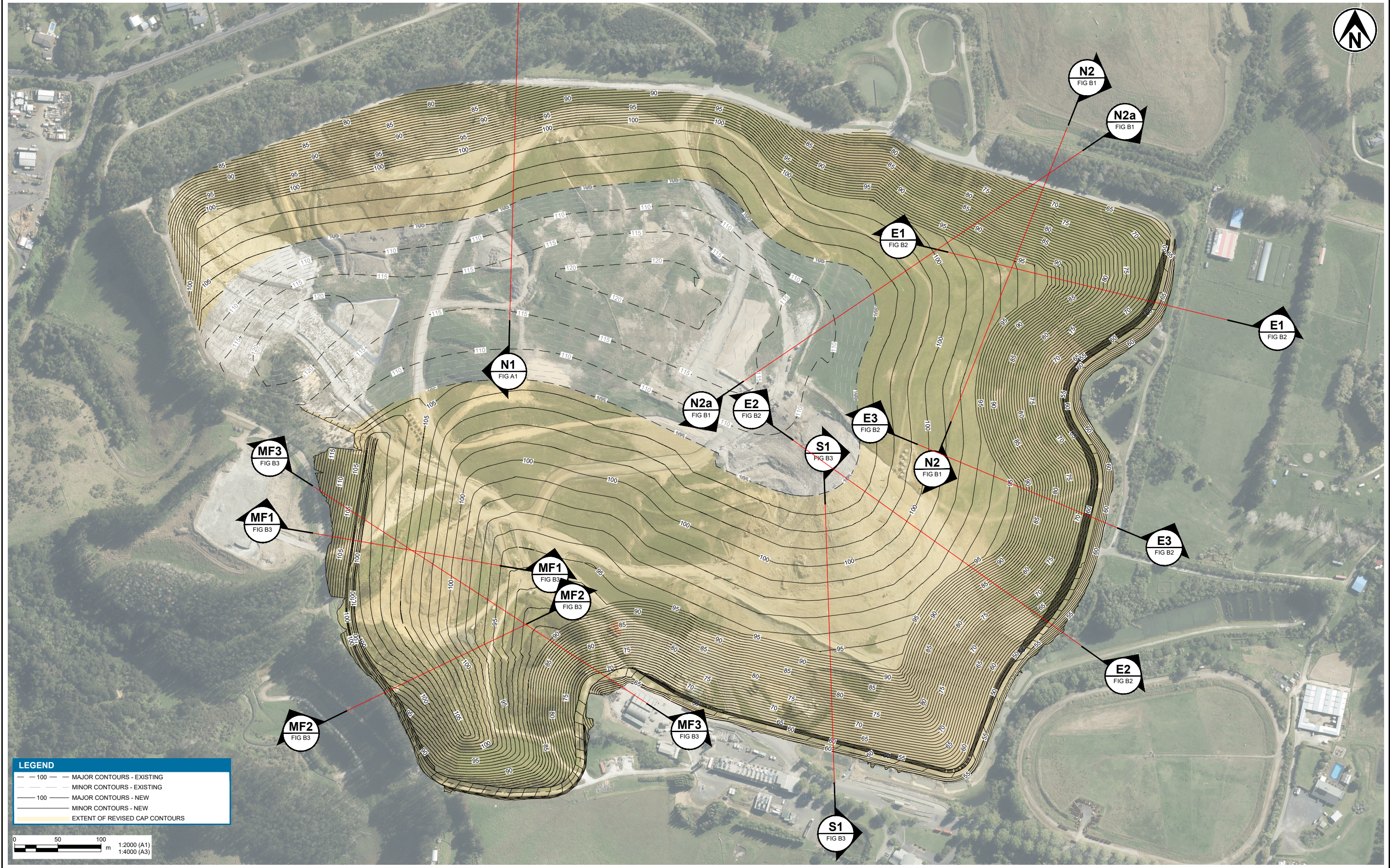
Andrew Stiles
Senior Principal Geotechnical Engineer

Appendix B Post-settlement contours plan and Basegrade plan

- B1 Key Plan**
- B2 Post-settlement contours plan**
- B3 Basegrade plan**
- B4 Site plan showing perimeter bund investigation boreholes**



	DESIGNED DRAWN DESIGN CHECKED FIGURE CHECKED	DES MELP DES. CAD	DATE JUN.25 DATE DATE	FIGURE STATUS PRELIMINARY DRAFT PROJECT PHASE CONSENT DESIGN	CLIENT WM NEW ZEALAND				
						PROJECT ARL TP			
						TITLE KEY PLAN			
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APPROVED					DATE		SCALE (A1) 1:2500	FIG No. FIGURE B1	REV 1



NOTES 1. FOR CROSS-SECTIONS SEE APPENDIX B 2. REFER TO TEXT OF GEOTECHNICAL ASSESSMENT REPORT, JUNE 2025 FOR DISCUSSION		DESIGNED KJUN DRAWN MELP DESIGN CHECKED FIGURE CHECKED	APR.25 MAY.25	FIGURE STATUS PRELIMINARY DRAFT PROJECT PHASE CONSENT DESIGN	CLIENT WM NEW ZEALAND PROJECT ARL TP TITLE POST SETTLEMENT CAP CONTOURS AND CROSS SECTIONS
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APPROVED		DATE			



LEGEND

—110

MAJOR CONTOURS - EXISTING

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MINOR CONTOURS - EXISTING

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MAJOR CONTOURS - NEW

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MINOR CONTOURS - NEW

EXTENT OF BASEGRADE EXTENSION



- NOTES
1. FOR CROSS-SECTIONS SEE APPENDIX B

2. REFER TO TEXT OF GEOTECHNICAL ASSESSMENT REPORT, JUNE 2025 FOR DISCUSSION

DESIGNED	KJUN	APR.25	FIGURE STATUS PRELIMINARY DRAFT PROJECT PHASE CONSENT DESIGN
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APPROVED		DATE	

CLIENT	WM NEW ZEALAND
PROJECT	ARL TP
TITLE	BASEGRADE CONTOURS
SCALE (A1)	1:2000
FIG No.	FIGURE B3
REV	1

