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SLOPE STABILITY GEOTECHNICAL GUIDANCE SERIES

UNIT 7B.1 - AUCKLAND

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AUTHORS

Heather Lyons – ENGEO
Stephen Parkes – Tetra Tech Coffey
Aaron George – WSP

STEERING COMMITTEE

Richard Justice – ENGEO, NZGS (Project Lead)
Eleni Gkeli – Stantec, NZGS immediate past Chair
Ross Roberts – Auckland Council, NZGS
Philip Robins - Beca, NZGS Chair
Kiran Saligame, MBIE
David Stewart, WSP

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Natural Hazards Commission
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New Zealand Geotechnical Society (NZGS)
PO Box 12 241
Wellington 6013

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The contents may be subject to changes, additions, and deletions. Submissions by the geotechnical community to the Society are encouraged, as it is intended that the guidance will be updated from time to time.

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CONTENTS

GLOSSARY

Explanatory Note on NZGS and MBIE Referencing:

This document refers to the New Zealand Geotechnical Society (NZGS) Slope Stability Guidance Series (of which this document is Unit 7B.1). The units are referenced in the text by their unit number: Unit 1, Unit 2 and so on.

The preceding units should be consulted in conjunction with this document. They are:

Unit 1: General Guidance
Unit 2: Landslide Recognition, Identification and Field Investigations
Unit 3: Slope Stability Analysis
Unit 4: Mitigation Strategies for Slope Instability
Unit 5: Rockfall Assessment, Analysis and Mitigation
Unit 6: Debris Flow Assessment, Analysis and Mitigation
Unit 7: Special Cases and Materials

The following abbreviations are used in this document:

- **ACCoP:** Auckland Council Code of Practice
- **AGS:** Australian Geomechanics Society
- **AUP:** Auckland Unitary Plan
- **CPT:** Cone Penetration Test
- **CU:** Consolidated Undrained Triaxial Compression Test
- **DCP:** Dynamic Cone Penetrometer
- **DMT:** Dilatometer Test
- **Fmtn:** Formation
- **GNS:** GNS (Geological and Nuclear) Science
- **LEM:** Limit Equilibrium Method
- **LL:** Liquid Limit
- **MBIE:** Ministry of Business, Innovation and Employment
- **MSE:** Mechanically Stabilised Earth
- **NZGS:** New Zealand Geotechnical Society
- **PC120:** Plan Change 120
- **PGA:** Peak Ground Acceleration
- **PI:** Plasticity Index
- **PL:** Plastic Limit
- **PSD:** Particle Size Distribution
- **RAMMS:** Rapid Mass Movement Simulation
- **SDMT:** Seismic Dilatometer Test
- **SPT:** Standard Penetration Test
- **UCS:** Uniaxial Compressive Strength
- **ULS:** Ultimate Limit State, as defined in NZS1170
- **UU:** Unconsolidated Undrained Triaxial Compression Test

1 INTRODUCTION

The Auckland Region is an important economic centre for New Zealand, accommodating 33.2% of the country's total population and having grown 5.4% since 2018[1]. This rapid growth places significant demand on land for residential and infrastructure development. The severe storm events of 2023 have underscored the regions vulnerability, with extensive land instability causing severe damage and loss of human life. Understanding the challenges and constraints to managing land instability in the Auckland Region is intrinsic to ensuring that the growth of the city is implemented without increasing landslide risk.

This document covers Slope Stability Guidance particular to the Auckland region. It is not intended to repeat information presented in the preceding units except where necessary for clarity but is intended to provide supplementary guidance to geotechnical practitioners and other professionals in recognising the geological constraints unique to Auckland's regional geology. The Northland Allochthon is introduced in this unit as it occurs in Auckland and is highly susceptible to slope instability. However, NZGS intends for Unit 7 to include standalone guidance specific to this geology due to its complexity. Terminology used in this document will remain consistent with Unit 1.

1.1 REGULATORY CONTEXT

Section 1.5 of Unit 1 presents a list of the national regulatory documents, standards, and guidance documents under which practitioners may operate (and indeed should have some familiarity with) in New Zealand. The following documents are relevant for practitioners undertaking slope stability work in Auckland:

- **Auckland Unitary Plan (AUP) Operative in part (updated 14 November 2025):** Sets out the statutory requirements for subdivision, infrastructure, land use and development activities in Auckland under the Resource Management Act. Natural hazards including "land which may be subject to land instability" are specifically discussed in Chapter E36, and other chapters refer to management of hazards including land instability in various contexts (e.g., E12, E26, E38, E39). Chapter J1 includes definitions for terms used in the AUP including a definition for "Land which may be subject to land instability" (see inset).
- **Plan Change 120: Housing Intensification and Resilience (notified 3 November 2025):** Currently open for submissions at the time of publication of this guidance, Plan Change 120 (PC120) proposes changes to the AUP Operative in part to better manage land development activities for natural hazards across the region, with new provisions for landslides (among other hazards). Although PC120 is open for submissions, some of the proposed plan change rules with respect to natural hazards have immediate legal effect in accordance with section 86B(3) of the Resource Management Act. Revisions to Chapters E12, E36, E38, E39 and J1 are proposed, and a landslide hazard risk assessment in accordance with new Appendix 24 is required for resource consent applications for proposals to subdivide, use or develop land within a landslide risk area.
- **The Auckland Code of Practice for Land Development and Subdivision (ACCoP), Chapter 2: Earthworks and Geotechnical (Version 2.0, May 2023):** Provides guidance on the minimum standards expected by Auckland Council for geotechnical design and construction of new assets to be vested in Auckland Council ownership. However, Auckland Council encourage its use for all land development projects as the content outlines the minimum standards expected to support resource or building consents..

Section 2.6.8 of this document sets out the reporting requirements for slope stability analysis, and the target minimum factors of safety for global stability. Where the designer proposes that a lower factor of safety is appropriate, the onus is on the designer to provide justification for that proposal through a risk assessment clearly demonstrating the likelihood and potential consequences of failure.

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[1] As based on census data from 2023 and reported on the Stats NZ website

The following general guidance documents published by Auckland Council also contain useful context for hazard assessment and design involving land instability in Auckland:

- Coastal hazard assessment in the Auckland region. Auckland Council Guideline Document, GD2021/010.
- Predicting Auckland's exposure to coastal instability and erosion, Auckland Council, Technical Report TR2020/021.
- Auckland Region Landslide Susceptibility Assessment. Auckland Council, Technical Report TR2025/7.
- Erosion and sediment control guide for land disturbing activities in the Auckland Region. Auckland Council, Guideline Document 2016/005.
- Stormwater soakage and groundwater recharge in the Auckland region. Auckland Council, Guideline Document GD2021/07.

These documents may be revised or superseded over time, and practitioners should check that they are using the most relevant and up to date guidance when undertaking this work.

AUCKLAND UNITARY PLAN OPERATIVE IN PART (UPDATED 14 NOVEMBER 2025)

SECTION J1: DEFINITIONS

LAND WHICH MAY BE SUBJECT TO LAND INSTABILITY

Any land with one of the following characteristics:

(a) Where the land which is underlain by Allochthonous soils has slope angles greater than or equal to 1 vertical to 7 horizontal;

(b) Where the land which is underlain by Holocene or Pleistocene sediments which has a slope angle greater than or equal to 1 vertical to 4 horizontal;

(c) Where the land is underlain by any other soil type and has a slope angle greater than or equal to 1 vertical to 3 horizontal;

(d) On sloping sites where fill greater than 600mm depth has been placed in uncontrolled conditions or not to engineered (certified) standards and where the original underlying natural terrain gradient was greater than or equal to:

i. 1 vertical to 7 horizontal for slope comprising Allochthonous soils;

ii. 1 vertical to 4 horizontal for slopes comprising Holocene or Pleistocene soils; or

iii. 1 vertical to 3 horizontal for slopes comprising any other soil types;

(e) Within a horizontal distance of 2.5 times the cliff vertical height behind the base of any natural cliff; or

(f) Within a horizontal distance of 2 times the cliff vertical height in front of the base of any natural cliff.

NOTE

A natural cliff may be considered to be any slope with a vertical height of greater than 3.5m and a gradient equal to or greater than 1 vertical to 1 horizontal (45-degrees). The vertical height of the cliff must only be measured over that part of the cliff where the slope gradient is equal to or greater than 45 degrees.

Geological conditions, including soil types not mapped in the Plan and soil conditions as referred to in the above definition may be identified at a regional level through the following sources:

- ***reference to information in GNS Sciences Qmaps;***
- ***Geology of Auckland (compiled by Edbrooke for GNS 2001);***
- ***property files material and reports held by Council; and***
- ***by a suitably qualified professional.***

2 AUCKLAND'S GEOLOGICAL SETTING

The Auckland Region exhibits significant geological complexity. Recent (in geological terms) volcanic activity, coupled with localised sedimentation and tectonic movement, has produced a diverse array of rock and soil types arranged in a highly intricate distribution. The complex geological history, diverse deposits and irregular topography creates numerous conditions conducive to slope failures. It is also evident that a correlation exists between specific rock or soil types and the likely modes of slope failure.

Any relationship of this sort may be a useful guide but by no means is it infallible. Existing geological and soil maps (see references) and other publications can be used to identify the types of rock mass and sometimes the soil mass, but these maps are not sufficiently detailed or of a suitable scale to be used for specific site investigation other than as an initial guide. An informed and purposeful investigation is essential for the assessment of the stability of slopes (Unit 2 has detailed advice). The geological formations which make up the region and the sorts of failure commonly associated with them are outlined later; however, the following four points should be considered:

1. Most slope failures occur in the layer of weathered soils overlying rock or in the recently deposited soft alluvial soils. Hard, dry, cracked surface clays are misleading, because they often overlie deeper layers of wet, weak material.
2. Soil creep is one of the most significant and overlooked types of slope failure. This relatively slow, almost imperceptible down-slope movement is brought about by changes in soil moisture content on slopes as gentle as 14° (1V:4H) and is also attributed to the high shrinkage capabilities of Auckland clays. Its effects are most serious when structures are built on shallow foundations on sloping sites.
3. Failures within the underlying rock masses are relatively uncommon. They do occur however and can be both spectacular and serious.
4. Generally, natural slopes steeper than 18° (1V:3H) warrant specific engineering design. Slopes less than 18° may also need special design and specialist advice, as set out in the AUP. Gentle slope angles do not necessarily mean that less stringent care is needed in founding buildings or undertaking earthworks. Slope failures can also occur when some very gentle, almost flat slopes are disturbed.

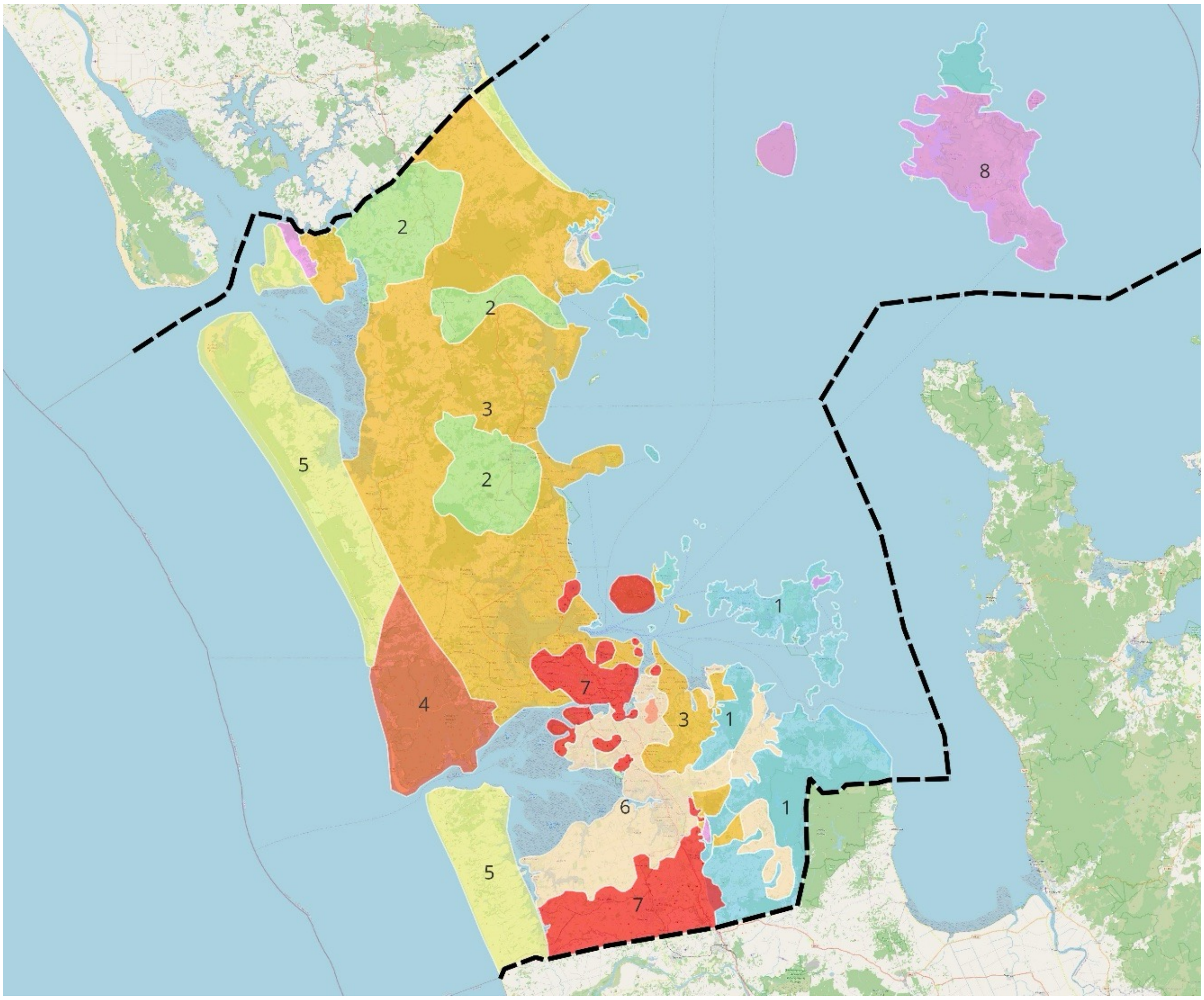


FIGURE 1: Broadly generalised geomorphic and geological regions in Auckland

For the purposes of this guidance the Auckland Region has been divided into geographical areas based on the prevailing geological unit and generalised geomorphic setting (Figure 1). The area boundaries are approximate and are only introduced to illustrate this discussion. The geological units described do exist outside of these areas.

These areas are:
1. Waipapa Group Terrane (principally Hunua Ranges and Waiheke Island)
2. Northland Allochthon
3. Waitematā Group
4. Waitākere Ranges (Manukau Subgroup, Waitākere Group)
5. West Coast Dune Sands (Awhitu and Kariotahi Groups)
6. Southern Lowlands (Takaanini Formation)
7. Auckland and South Auckland Volcanic Fields
8. Other Localised Formations

The following subsections describe the geological context encompassing each of these areas.

2.1 WAIPAPA GROUP TERRANE

2.1.1 GEOLOGY

The oldest basement rocks that outcrop in Auckland belong to the Waipapa Group Terrane, which range in age from Triassic to Early Cretaceous. These rocks comprise two distinct sequences: a deep-marine sequence and a terrigenous (land-derived) sequence, the latter of which dominates the present-day stratigraphy.

The terrigenous rocks comprise predominantly greywacke and some argillite (muddy siltstone and mudstone that has undergone a higher degree of compaction and hardening). The sequence was derived principally from the transport of sand and mud via turbidity currents off the margins of the ancient supercontinent of Gondwana, which was situated on the continental side of a convergent plate boundary (subduction zone).

Arc volcanism associated with the subduction zone is inferred as the principal source of clastic material (Hayward, 2017), which is also represented in the stratigraphy by the occurrence of rare thin tuff beds.

The older and less abundant deep marine units, laid down on the oceanic side of the subduction zone, comprise basaltic pillow lavas erupted from an oceanic spreading ridge, red cherts derived from the accumulation of marine siliceous zooplankton and green and grey argillites from the settling of suspended terrigenous silts and muds on the sea floor.

The two sequences have been emplaced together in packets at the plate boundary, where rafts of the oceanic units were accreted into the terrigenous greywackes and argillites forming an accretionary prism (wedge of highly deformed material at the boundary of two converging tectonic plates).

The high-stress environment of the accretionary prism and subsequent deep burial of the Waipapa Group units resulted in steeply dipping, highly deformed, tightly folded and faulted sequences. Less competent argillite beds are often highly sheared, and crushed zones within the rock mass and melange zones (highly deformed, undistinguishable 'mixed' sequences) are common. The sedimentary units (greywacke and argillite) are thinly to massively bedded and typically very closely fractured (see Figure 2).

The deep burial also resulted in low grade metamorphism of the Waipapa Group Terrane rocks (Black et. al, 1993), with frequent quartz, calcite, zeolite, chlorite and prehnite veins forming within the abundant fractures. The resultant rocks are generally harder than their non-metamorphosed sedimentary equivalents (typically strong to very strong in their unweathered state), which has led to the Waipapa Group greywackes being widely quarried for use as building aggregates.

The Waipapa Terrane units generally weather to yellow, brown, white and orange, occasionally mottled red and pink, stiff residual silts and clays to depths of typically less than 10 m. Weathering reduces towards the base of the soil mantle where the soils increasingly retain the predominantly dark blue grey colouration and structure of the parent rock. The weathering profile in Waipapa Terrane frequently extends deep into the rock mass with unweathered rock often intercepted at depths of 20 m or more.

2.1.2 GEOGRAPHICAL DISTRIBUTION

Waipapa Group Terrane rocks outcrop across the Hunua Ranges, as well as Great Barrier, Waiheke and various other Hauraki Gulf Islands, with some pockets around Leigh and surrounding areas.

2.1.3 GEOMORPHOLOGY

Geomorphologically, the Waipapa Group Terrane is characterised by high, steep jagged slopes and incised valleys typical of the Hunua ranges and Waiheke Island. The margins of mountainous Waipapa areas are often juxtaposed against geologically younger, gentler or low-lying landforms by faults.

Despite the high degree of fracturing, slopes are generally stable at angles of up to 30°, with instability predominantly occurring within the shallow residual soils situated within over steepened gullies. Long-term soil creep commonly influences the surficial soils on the steeply formed slopes.

In coastal areas and in steep exposed rock cuttings, due to the closely spaced joint sets, rockfalls, topples and wedge failures are common.

FIGURE 2: Typical closely fractured, weathered (left) and unweathered (right) rock core in Waipapa Group greywacke, Hunua Quarry

Where rock structure is preserved into the completely weathered or transitional soil layer, cuts or natural slopes exposing this soil layer may behave like weak blocky rock.

2.1.4 GROUNDWATER

Due to the generally high relief of Waipapa Group Terrane hill country and high degree of fracturing serving to increase permeabilities, groundwater tables are generally intersected at depth within the Waipapa Group Terrane.

2.2 NORTHLAND ALLOCHTHON

The Northland Allochthon is covered in detail in a separate standalone guidance unit (Unit 7A.1, in preparation). The following is intended to provide a brief overview only, and Unit 7A.1 should be consulted for a detailed geotechnical summary of its stability characteristics and risk mitigation strategies.

2.2.1 GEOLOGY

The Northland Allochthon, previously named Onerahi Chaos Breccia, was emplaced into the Northland and Auckland Regions during the early to mid-Miocene as a series of thrust sheets of oceanic sediments and crustal materials. Units of the Northland Allochthon were subsequently deposited adjacent to and within the sediments of the Waitematā Group that were deposited in the Waitematā Basin (Hayward, 2017).

Hayward's Chapter 4 "Northland's Displaced Rocks" provides a comprehensive description and model of the Northland Allochthon together with useful commentary on how this complex suite of rocks came to be understood.

To summarise Hayward, giant slabs of sedimentary rocks northeast of present-day Northland were tectonically uplifted and translated hundreds of kilometres southwest into their present position in Northland and northern Auckland in the Cretaceous through mid-Tertiary (c. 100-40 million years ago).

The Northland Allochthon has a highly sheared and fractured, chaotic structure, with a weak-moderately strong intact material strength but low rock mass strength in an unweathered state. The materials include very weak to weak calcareous and non-calcareous mudstone, weak to moderately strong fine-grained limestone, and displaced volcanic massifs. Due to the emplacement of the Northland Allochthon materials, the tectonic disturbance manifests in a highly complex tectonic overprint that can dominate the rock materials. The tectonically deformed rock is best described as melange, fault breccia and fault gouge. In general the Allochthon can be distinguished by its finely fractured and sheared texture with innumerable small, polished faces and grooved striated surfaces (i.e., slickensides).

Residually weathered Allochthon soils typically form a thin cohesive mantle (2 to 4 m) with no remaining rock mass fabric. As the degree of weathering decreases with depth and the rock mass fabric is better preserved, the behaviour of the soil layer is primarily controlled by the relict defects and preferential shear surfaces develop. Some residually weathered clays are highly expansive and are susceptible to substantial volume changes.

The pervasively sheared fabric is not limited to the main displaced Cretaceous to Oligocene units of the Allochthon. At the southern intersection with the Miocene Waitemata Group, rafts of Waitemata Group units appear to have been picked up and displaced with the Allochthon as it moved southwards. As such, contacts between the two are intensely sheared and the Waitemata Group rocks (principally Pakiri and East Coast Bays Formations) exhibit the similar chaotic texture to the Allochthon and may be better termed "Allochthonous Waitemata Group" to distinguish them from their in-situ equivalents further south. The stability hazard associated with developments in these geologies are perhaps better understood in the context of the Northland Allochthon than the parent Waitemata Group.

2.2.2 GEOGRAPHICAL DISTRIBUTION

The two high-level Allochthon units present in the northern Auckland area are the Mangakahia and Motatau Complexes, defined by the age of the of original deposition of the sedimentary rocks (dated by fossil evidence, particularly micro-fossils). Each of these complexes is split into a number of formations by lithology (Hayward's stratigraphy differentiates more of these lithological units than Edbrooke, as it covers the full extent of the complexes across the Allochthon).

Hayward describes the internal structure of one part of the allochthon (Whangaroa-Kaitaia in the "Far North" of Northland), where exposure is better, as *"many of the slabs (especially of Mangakahia and Motatau Complexes) … consisted of a number of smaller sheets sheared against each other. The contacts between the slabs were often not visible, but when seen consisted of a melange of various rock types in a matrix of sheared clay-rich mudstone (Hukerenui Mudstone) which presumably helped lubricate the slowly-moving slabs as they slid in."*

Echoing Edbrooke, Hayward also notes "*It appears that the extensive displaced slabs of northern Northland were dismembered into smaller and more sheared blocks as the southern edge of the Allochthon advanced progressively towards Auckland.*"

The Allochthon boundary with the Pakiri Formation of the Waitematā Group just north of the Hoteo River (south of Wellsford), is considered the southern front of the continuous Northland Allochthon. Localised Allochthon occurrences south of this point between Warkworth and Silverdale are interpreted as the result of giant seafloor failures of allochthon materials sliding into the actively subsiding Waitematā Basin.

2.2.3 GEOMORPHOLOGY

The allochthonous landforms are expressed geomorphically as low-lying hills, that are associated with hummocky terrain and low relief. Surface water and ponding, hydrophilic vegetation, and terracettes are commonly observed.

The issues the Northland Allochthon pose for landslides are significant (George & East, 2001). Typically, two types of failure occur within the geology: natural slope failure and cut slope failure. Large-scale slow moving natural landslides are prevalent. Graben-style terracettes can occur below the head scarp area, which is part of the ongoing "slope flattening" process. Natural slope failures develop as ongoing intermittent failures, often discreetly bulging at the toe downslope forming mounds (Hayward, 2017).

Natural failures in undeveloped areas are typically driven by groundwater. Cut slopes in contrast can fail on excavation due to stress release in the sheared rock mass resulting in dilation and ravelling of the material upslope. Cut and fill slopes associated with roading and building development in north Auckland typically require active management measures to mitigate instability associated with the change in slope loading. Caution is required where cuts expose the pervasively sheared rock mass or less weathered soil layers as the introduction of surface water into that fabric can initiate failure downslope or trigger large-scale or localised translational failures of slopes above.

Natural slopes are typically undulating and gentle. The sheared mudstones are rich in clays, and in the presence of groundwater the clays readily lubricate the movement of landslides, particularly in those formations that break down to smectite rich clays (e.g., Hukerenui Mudstone).

2.2.4 GROUNDWATER

High groundwater levels are found associated with landslide features that can be identified by the presence of localised springs, and accompanying hydrophilic vegetation.

Elevated pore water pressures, occasionally artesian, can develop in the soil-rock transitional zone and upper highly fragmented rock mass during winter months due to the outer residual soil layer acting as a low permeability cap.

FIGURE 3: Examples of natural landsliding in Northland Allochthon terrain.

2.3 WAITEMATĀ GROUP

2.3.1 GEOLOGY

Subsidence and a resulting marine transgression in the early Miocene led to the formation of the Waitematā Basin. The sediments that infilled this basin are the Waitematā Group, which lie unconformably above older stratigraphic units (Waipapa Group Terrane and Tu Kuiti Group).

In a regional context, the basin in the early Miocene was situated in close proximity to a convergent plate boundary (subduction zone), which included two lines of associated arc volcanoes located offshore to the west and east of the present-day North Island. To the north, the rocks of the Northland Allochthon were being thrust southwards via a series of giant mobilised slabs (Hayward, 2017).

These two sources (arc volcanism and the southward emplacement of the Northland Allochthon) provided a supply of clastic material into the Waitematā basin for 8 to 10 million years (Hayward, 1993).

The Waitematā Group rocks are a series of principally deep marine turbidite (also known as flysch) sedimentary sequences with variable volcanic content, up to 1000 m thick, which represent the remobilisation of the clastic material onto the sea floor.

Various distinct formations are present (and summarised in Table 1) from shallow marine sequences representing the initial onset of marine conditions (Kawau Subgroup), to deep submarine channel conglomerates (Albany, Helensville, Matapoura Conglomerates) and volcaniclastic grits (Parnell Grit).

The most widespread formations comprise interbedded fine to medium grained sandstone and mudstone sequences, which were deposited in low energy currents on the outer lobes of the submarine fans.

Structurally, the Waitematā Group units tend to be gently dipping but locally can be highly deformed into near vertical beds or tight, recumbent folds. Bedding dips can increase locally near faults, which frequently intersect the Waitematā Group units, often forming grabens and half grabens which shape the surficial topography and geomorphology. Flexural slip (i.e., bed-over-bed sliding) during folding has led to the development of clay seams up to approximately 30 mm thick along bedding planes that can be traced across sites by correlation with marker horizons (Williams and Prebble, 2004).

The rocks encountered are typically very weak to weak, but rock strengths generally increase with increasing volcanic content. They weather to a residual soil mantle consisting of typically stiff or very stiff orange and grey silty clays. The interface between soil and rock transitions through a zone of slightly less weathered soil comprising higher shear strengths and increased granular soil content. The thickness of the 'transition zone' layer is governed by the intensity of the weathering of the underlying rock mass, and a gradual or abrupt weathering profile has been described by Wesley (1988) and Adhikary (2001). In a gradual weathering profile, the residual soil is completely structureless in keeping with NZGS (2005). In an abrupt weathering profile, the transition zone is relatively thin, and the overlying soil retains some of the bedding and structure of the underlying rock mass. The weathering profile has a direct influence on the behaviour of a slope, and its geomorphic expression.

2.3.2 GEOGRAPHICAL DISTRIBUTION

The Waitematā Group is exposed across much of the central and northern parts of the Auckland Isthmus, extending as far south as Drury and north of the Auckland urban area.

Table 1: Waitematā Group Geology Summary Table

Main Subgroup	Formations and Rock Types	Geological Origin	Approx. Age (Ma)	Geographic Distribution
Kawau Subgroup	**Shallow Marine Rocks** • *Cape Rodney Formation* (Greywacke conglomerate and breccia, lithic sandstone, occasional limestone)	Flows of terrigenous (land-derived) sediments into shallow water. Represents the Initial on-set of the marine Waitematā basin. Some shallow marine limestones.	21	• *Cape Rodney, Kawau Island, Leigh, Tauwharanui Peninsula, Waiheke Island*
Warkworth Subgroup	**Deep Marine Turbidites:** • *East Coast Bays Formation* (volcanic-poor fine to medium grained sandstone and mudstone) • *Pakiri Formation* (volcanic-rich fine to coarse grained sandstone and mudstone) • *Timber Bay Formation* (volcanic-rich interbedded fine-grained sandstone and mudstone) • *Blockhouse Bay Formation* (interbedded volcanic poor and volcanic-rich sandstones and mudstones) • *Cornwallis Formation* (Interbedded fine to coarse grained sandstone and siltstone with variable volcaniclastics) • *Hoteo Beds* (interbedded fine sandstone and mudstone)	Distal turbidite deposition at bathyal depths (≈1000 to 2000 m below sea level). Variable volcanic content represents variations in the spatial proximity to volcanic centres and periodic increases and decreases in volcanic activity.	17-20	• *East Coast Bays Fmtn:* Widespread between Silverdale and Drury • *Pakiri Fmtn:* From west to east coasts from Orewa to north of Wellsford • *Timber Bay Fmtn:* Kaipara Area • *Blockhouse Bay Fmtn:* West of Auckland city between Kumeu and Manukau Harbour • *Cornwallis Fmtn:* West Auckland from Manukau Harbour to Kaipara Harbour • *Hoteo Beds:* Between Warkworth and Wellsford
	Marine Turbidite Channel Lithologies: • *Albany Conglomerate* (Conglomerate of gravel to boulder sized clasts in a medium to very coarse-grained sand matrix) • *Helensville Conglomerate* (Conglomerate of gravel to boulder sized clasts of igneous rocks in a medium to very coarse-grained sand matrix) • *Matapoura Conglomerate* (Conglomerate of gravel to boulder sized clasts of igneous rocks in a medium to very coarse-grained sand matrix)	Coarse particle sizes indicative of higher energy deposition within the main turbidite channel. Constituent clasts represent the remobilised remnants of material emplaced within the basin including Tangihua Complex rocks (Albany and Matapoura Conglomerate) and Andesitic Volcanics of the Kaipara Volcano (Helensville Conglomerates).	17-20	• *Albany Conglomerate:* NW of Albany, Riverhead Forest • *Helensville Conglomerate:* North of Waimauku to Kaipara Harbour • *Matapoura Conglomerate:* Kaipara Harbour
	Other Lithologies: • *Parnell Grit Beds* (0.5 to 20m thick volcaniclastic graded sandstone layers interfingered within the East Coast Bays and Pakiri Formations)	Lahars (volcanic debris flows) that flowed into the marine basin following eruptions of the Manukau (Waitākere) and Kaipara volcanoes.	17-20	Widespread across the Auckland region
Kaipara Subgroup	**Deep Marine Turbidites:** • *Waihangaru Formation* (Predominantly thin bedded fossiliferous mudstone and some fine sandstone)	Distal turbidite deposition at bathyal depths (≈1000 to 2000 m below sea level).	17-18	Okahukura Peninsula, Kaipara Harbour

FIGURE 4: Block sliding within East Coast Bays Formation rocks at Granny's Bay, Long Bay Regional Park: The extensional crustal forces that formed the Granny's Bay Fault (Blue dashed line) also produced three distinct low-angle, bedding parallel shear zones (red dashed lines) that have strongly influenced the geomorphology of the upslope land via a series of block slides. Note groundwater seepage and vegetation growth from the shear zones in the cliff exposure.

FIGURE 5: A tilted block of East Coast Bays Formation in the Southern Landslide Zone, demonstrating broad slopes parallel to bedding (A) and steep back-bedding geomorphology (B). Localised shallow-seated instability geomorphology is evident on the steeper slopes (C).

2.3.3 GEOMORPHOLOGY

Waitematā Group terrain is generally characterised by moderately steep hill country, with natural slope angles typically increasing in the coarser geological formations (e.g. Albany Conglomerate, Parnell Grit). Ridgelines, gullies and coastal cliff alignments are strongly structurally controlled by dominant NNW-SSE and NE-SW trending regional lineaments.

Over-steepened slope portions bound by concave head scarps and hummocky ground are common, indicating terrain shaped by slope instability processes. Theses geomorphic features are typically characteristic of shallow-seated instability associated with elevated porewater pressure at the contact between layers of higher and lower permeability.

Undulating landforms that step down from higher to lower elevations via a series of steeper slope portions and gentler benches are indicative of larger block slides, with the base of steeper sections often aligned with gently dipping bedding-parallel shear planes or clay seams (Figure 4).

The Southern Landslide Zone is a region of unstable land identified in the 1:50,000 geological map for Auckland (Kermode, 1992) and is a type locality for land impacted by parallel bedded clay seams. The area broadly extends over the East Tamaki Heights and Whitford areas, south to where the surface expression of the East Coast Bays Formation abuts the Waipapa Group north of Brookby, and dives below the Takaanini Formation in Alfriston. This region is characterised by large, deep-seated block slide geomorphology including head scarps, landslide blocks and mid-slope benches, and irregular stream morphology where landslides have altered their course. Debris mounds, areas of hummocky ground, debris flow lobes, and swampy ground indicative of groundwater springs are also prevalent.

2.3.4 GROUNDWATER

Two groundwater profiles are generally encountered in the Waitematā Group; shallow groundwater trapped within the near-surface low-permeability residual soils and the deep regional groundwater table often encountered at depth within the underlying rock. Low permeability interfaces at the base of the soil mantle can provide barriers to the vertical flow of groundwater, resulting in pore pressure development and associated translational instabilities. Excavations that penetrate these interfaces often encounter them as springs, with the preferential flow of water extending laterally into the excavation.

2.4 WAITĀKERE RANGES (MANUKAU SUBGROUP, WAITĀKERE GROUP)

2.4.1 GEOLOGY

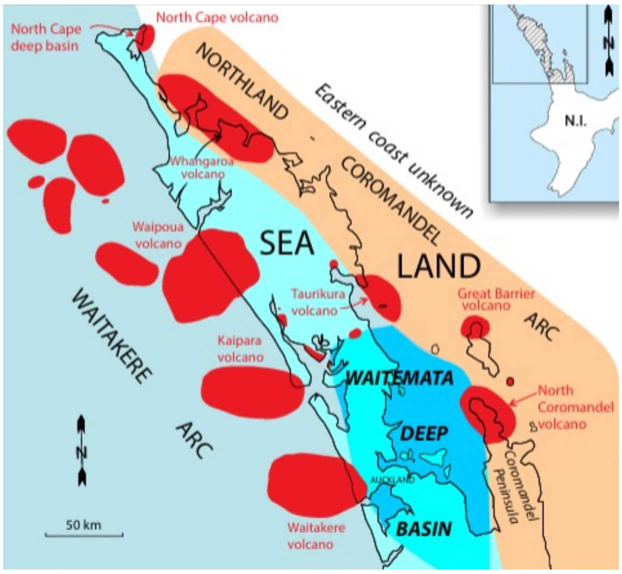
Offshore geophysical anomalies and geological observations across the Waitākere Ranges indicates the presence of a now extinct, eroded volcanic cone located off the present-day west coast.

This stratovolcano, known as the Manukau or Waitākere volcano, was active throughout the early Miocene and released large quantities of volcanic debris which flowed into the marine sedimentary basins to the east.

The rocks of the Manukau Subgroup, as they are now known, form the Waitākere ranges, and consist of several distinct lithologies composed of varying quantities of volcanic material governed by their proximity to the now extinct volcanic edifice. These formations are summarised in Table 2. Their distribution across the area is shown in Figure 6.

2.4.2 GEOGRAPHICAL DISTRIBUTION

These formations form the Waitākere Ranges and crop out west of Waitākere Township from Muriwai in the north to Manukau Harbour in the south, with a small outcrop also visible at Clarks Beach.



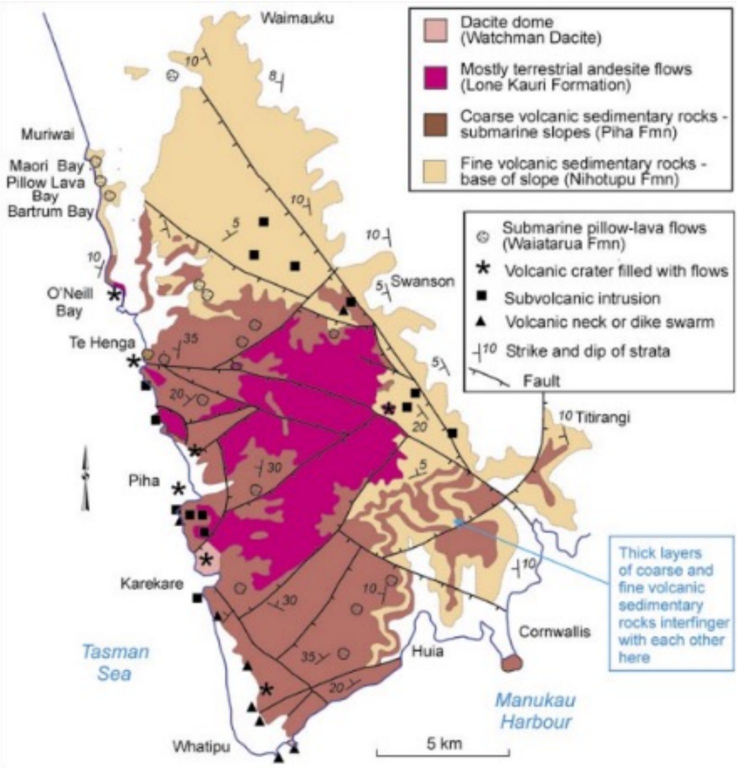


FIGURE 6: Left: Northern New Zealand approximate geography 20 million years ago (image taken from Ballance, 2009)
Right: Present-day distribution of Manukau subgroup rocks across the Waitākere region (top), and their relationship to the now extinct volcanic centre (bottom) (images taken from Hayward, 2017).

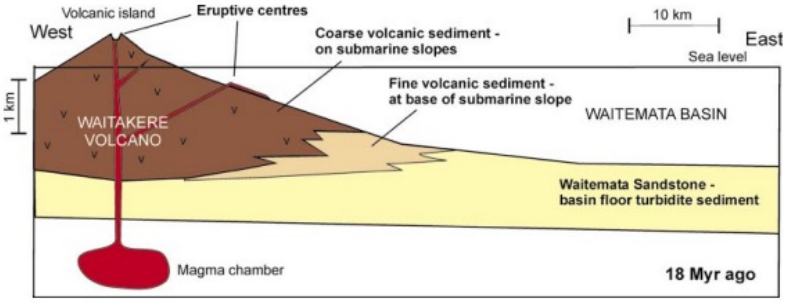


Table 2: Summary of Manukau Subgroup, Waitākere Group geological units

Main Subgroup	Formations and Rock Types	Geological Origin	Approximate Age (Ma)	Geographic Distribution
Manukau Subgroup	**Deep Marine Turbidites:** • *Nihotupu Formation (fine to coarse grained volcaniclastic sandstone and siltstone)*	Distal turbidite deposition at bathyal depths (≈1000 to 2000 m below sea level). Volcaniclastics principally the remobilised volcanic material that was accumulating on the flanks of the volcanic centre to the west.	16-18	Northern and southern parts of the Waitākere Ranges
	Lava Flows and Volcanic Debris Avalanches • *Piha Formation (Basaltic and Andesitic conglomerates and breccias, interbedded medium to coarse grained sandstone turbidites, rare tuff beds)*	Debris avalanches of rapidly accumulating volcanic material. Some turbidite sandstone sequences but finer siltstones are absent.	16-18	Widespread across the Waitākere Ranges
	Lava Flows and Intrusive Igneous Rocks • *Waiatarua Formation* (Sheets of basaltic andesite dykes and sills, effusive lava flows and pillow lavas, volcanic breccias) • *Lone Kauri Formation* (Sheets of basaltic andesite dykes and sills, effusive lava flows and pillow lavas, volcanic breccias)	Intrusive magmatic sheets and effusive lava flows from volcanic vents protruding eastward into the marine basin.	16-18	• *Waitarua Fmtn:* Pockets outcrop across the Waitākere Ranges • *Lone Kauri Fmtn:* Widespread across the Waitākere Ranges, forms most of the highest elevation hill slopes
	Channel Deposits • *Tirikohua Formation* (Volcaniclastic conglomerate, pebbly sandstone and fine-to-coarse grained sandstone)	Three wide submarine channels carved into the underlying Nihotupu Formation. Remobilised older Waitākere Group units via debris avalanche.	16-18	South of Muriwai Beach

2.4.3 GEOMORPHOLOGY

The geological units of the Waitākere ranges generally weather to a surficial soil layer less than 10 m thick of orange, brown, red and occasionally purple stiff silts and clays with occasional gravels. Sand content tends to increase towards the base of the weathered soil mantle. Beneath the surficial soil layer, Manukau subgroup rocks tend to be highly weathered to depths of 20 m (Edbrooke, 2001).

The volcanic origin of the Manukau Subgroup has resulted in high strength rocks able to stand at steep slope angles. As such, these geologies have given rise to the steep high slopes and incised valleys characteristic of the Waitākere Ranges, and rugged western coastline.

Due to the highly dynamic environment in which the geological units were formed, they are broadly heterogeneous. Subsequent tectonic deformations have therefore had a non-uniform effect on the stratigraphic sequences, and as with in the older Waitematā Group, flexural slip has resulted in the sporadic occurrence of low strength, planar sheared surfaces, shear zones or clay seams, parallel with bedding planes or lithological boundaries. This is particularly notable in the more distally deposited finer grained turbidite sequences (Nihotupu Formation), which generally are more susceptible to translational landsliding than the coarse, more volcanic-rich Piha, Waiatarua, Tirikohua and Lone Kauri Formations.

In coastal areas and in steep cut slopes, rock falls are reasonably common, but the more volcanic-rich units are typically highly resistant to weathering.

2.4.4 GROUNDWATER

The permeability of the Manukau Subgroup formations is highly variable, which can give rise to translational landslides when elevated pore water pressures develop on lower permeability horizons at the base of the soil mantle.

This risk is elevated in close proximity to faults, where the vertical permeability through the soil can increase locally, the likelihood of occurrence of lower permeability bedding parallel sheared surfaces is elevated, and the shear strength parameters of soil and rock layers intersected by the fault are reduced.

Near surface perched groundwater tables are common in the Manukau Subgroup, owing to the fine-grained soil mantle and range of material permeabilities.

2.5 WEST COAST DUNE SANDS (AWHITU & KARIOTAHI GROUPS)

2.5.1 GEOLOGY

Tectonic uplift has resulted in the Auckland area remaining predominantly above sea level since the mid-Miocene. Alluvial transportation of the eroded remnants of the uplifted rocks subsequently accumulated on west coast areas.

From the late Pliocene to Holocene, sea-level variations in response to glacial cycles and periods of active volcanism resulted in periodic influxes in transported sediments to these areas, which via the process of northward trending longshore drift, fanned out into long sand barriers up Auckland's West Coast.

The dune sands (Awhitu and Kariotahi Groups) comprise predominantly fine to coarse grained variably cemented often iron stained cross bedded sands of thicknesses of up to 200 m. Individual dune sequences range in thickness from metre to tens of metre scale and are often bound by clay-rich erosional contacts or organic rich paleosols and lignite correlative with glacial cycles (Hayward, 2017).

2.5.2 GEOGRAPHICAL DISTRIBUTION

Today, these units can be observed forming the North and South Kaipara barriers, extending from Hokianga Harbour to Muriwai Beach, and the Awhitu Peninsula from Manukau Heads to Port Waikato.

2.5.3 GEOMORPHOLOGY

Geomorphically, the Awhitu and Kariotahi Groups tend to form steep high slopes and incised valleys which generally remain stable at gradients of up to 1V:1.5H due to the influence of cementation. Coastal outcrops and road cuttings across the Awhitu and South Kaipara Peninsulas are indicative of this, often standing at notably steeper gradients than the friction angles of their uncemented constituent soil particles.

When exposed at ground level, the uppermost few metres of Awhitu Group and Kariotahi Group soils generally weather to silts and clays.

Slope failures within Awhitu and Kariotahi Group soils tend to be either rainfall-induced debris flows, or due to the largely uniform nature of the soils, rotational landslides. They are particularly vulnerable to concentrated overland flows, where progressive erosion and removal of sand particles causes localised oversteepening and subsequent failure.

FIGURE 7: Large slope failure, colluvium deposition and terracettes in Kariotahi Group Sands, Awhitu Peninsula

A mechanism for translational type landsliding in the west coast dune sands has been proposed, resulting from rapid pore water pressure increases at the low permeability basal contact of the Awhitu Group soils, and underlying Miocene geologies (Brook & Nicoll, 2024).

Due to the destruction of cementation, colluvial deposits that accumulate at slope toes are of significantly lower strength than their in-situ equivalents. The colluvium is therefore highly susceptible to retrogressive slope failure.

Terracettes are abundant on high Awhitu and Kariotahi Group hillslopes, suggesting surficial layers are affected by long term creep movement.

2.5.4 GROUNDWATER

The regional groundwater table within the Awhitu and Kariotahi Groups is generally intersected at depth due to the high hillslope topography, free draining soil and sparsity of thick low permeability clay and organic layers which tends to inhibit the development of perched groundwater tables. Where clay or organic-rich horizons are thicker however, perched groundwater tables can be encountered.

2.6 SOUTHERN LOWLANDS (TAKAANINI FORMATION)

2.6.1 GEOLOGY

The Takaanini Formation comprises Late Pliocene to Holocene, poorly consolidated sediments unconformably overlying the older geological strata across Auckland (Bland et. al, 2023). These deposits are associated with deposition in terrestrial and shallow marine environments and are locally interfingered with the volcanic units (Kerikeri Volcanic Group) and coastal dune sands (Awhitu and Karioitahi Groups) through central, south and south-western Auckland.

Formerly named Puketoka Formation and Tauranga Group, these sediments have been reclassified in an Auckland context by Barrell et. al. (2021) to allow for more precise characterisation and better geotechnical utility of the naming convention. The Takaanini Formation has been divided into eight geologically distinct lithofacies based on their dominant lithology and depositional environment, and as such the geotechnical characteristics that govern their behaviour. These lithofacies are summarised in Table 3 below.

Table 3: Takaanini Formation Lithofacies and their Geotechnical Characteristics, after Barrell et al. (2021)

Member	Short Description
Puketoka Gravel Member	Sandy or muddy gravel (conglomerate) with clasts from Coromandel volcanic rocks and Waipapa Terrane; may include volcaniclastic layers, organic material, or shells. Deposited in fluvial to estuarine settings.
Pahurehure Member	Fluvial sediments from local catchments; variable grain size (gravel to silt/clay); may include volcanic or carbonaceous material. Found in alluvial fans, stream valleys, or deltas.
Otahuhu Member	Shelly sand(stone), silt(stone), or mud(stone) with gravel and carbonaceous material. Deposited in estuarine to shallow-marine environments. Includes Holocene shell-rich deposits.
Curlew Member	Fine-grained sand(stone), silt(stone), or mud(stone) with possible gravel, clay, carbonaceous, or shelly material. Associated with lacustrine, estuarine, or shallow-marine settings.
Runciman Member	Carbonaceous sand(stone) or mud(stone) with prominent but not dominant organic matter. Occurs in fluvial, estuarine, lacustrine, or shallow-marine settings.
Ardmore Member	Peat (lignite) deposits dominated by organic material. Found in swamp environments.
Hobsonville Member	Silicic volcanic deposits (tephra, ignimbrite, reworked pumice); may include carbonaceous material, gravel, sand, and silty/clay. Deposited in colluvial, fluvial, or lacustrine settings.
Kepa Member	Landslide debris (breccia) with highly variable composition, grain size, and texture. Deposited in hillslope or hillslope-margin settings.

2.6.2 GEOGRAPHICAL DISTRIBUTION

Takaanini Formation sediments are widespread across the Auckland region. They underlie much of the Manukau Lowlands, mapped extensively at the surface on the eastern and southern sides of the Manukau Harbour. Deep peat swamp deposits extend to the east from Takanini through Ardmore and Clevedon out to the mouth of the Wairoa River. The western and north-western sides of the Waitematā Harbour are underlain by Takaanini Formation sediments, which unconformably overlie the Waitematā Group hills to the north and west.

Cliffs at Beachlands on Auckland's east coast feature Takaanini Formation soils unconformably overlying dramatically folded and faulted Waitematā Group strata (Figure 8). Slope instability is well documented in this area, particularly in locations where the Takaanini Formation sediments have infilled paleovalleys in the underlying rock.

2.6.3 GEOMORPHOLOGY

Sediments of the Takaanini Formation are typically associated with the subdued topography of the Manukau Lowlands and south of Manukau Harbour. However, the varied lithofacies result in a range of surface expressions including broad slopes and alluvial terraces, and low coastal cliffs.

The northern and western sides of the inner Waitematā Harbour feature low height cliffs (typically less than 10 m) consisting of primarily cohesive soils with some pumice content (e.g., at Hobsonville which is the

FIGURE 8: Takaanini Formation unconformably overlies tightly folded Waitematā Group at Sunkist Bay, Beachlands

reference locality for the Hobsonville Member). At Beachlands, these soils form the upper portion of the cliffs. Instability in these cliffs typically occurs as a translational slide on layers within the soil profile or at the basal contact with the underlying Waitematā Group strata. Rotational slides can occur where homogeneity of the soil mass allows.

Also captured in the Takaanini Formation stratigraphic framework is the Kepa Member. The Kepa Member replaces the previously named "Colluvium" map unit assigned to the Tauranga Group (Kermode, 1992), encompassing landslide deposits of variable composition having undergone destructive displacement (rather than block sliding of intact blocks). The Kepa Member is mapped at the ground surface in the recently published geological map for Pukekohe (Bland et. al., 2023), most notably through south-eastern Auckland overprinting the Waitematā Group and Waipapa Group rocks, in close proximity to the Bombay, Drury and Hingaia faults. It is also mapped in the South Auckland Volcanic Field hill country in and around Pukekohe in discrete areas, and on the coastal margins. New geological maps using the revised stratigraphic framework for Takaanini Formation have not yet been published for the wider Auckland region, however the Kepa Member is expected to hold a prominent place in the surface mapping.

A geomorphological map of the Pukekohe area to accompany the recently issued geological map and 3D geological model is in preparation (Townsend et al., in prep.).

2.6.4 GROUNDWATER

Groundwater in the Takaanini Formation is highly variable and directly controlled by the relationships between the granular, water-bearing lithofacies and the relatively impermeable cohesive or weakly cemented rock units. Perched water tables are common, and groundwater springs are observed where layer contacts daylight in slopes or cliff faces.

2.7 AUCKLAND AND SOUTH AUCKLAND VOLCANIC FIELDS

2.7.1 GEOLOGY

The Auckland and South Auckland Volcanic Fields form part of the Kerikeri Volcanic group and result from intraplate back-arc volcanism, active from the Early Pleistocene through to present day, with the most recent eruption at Rangitoto some 600 years ago. Volcanics of the South Auckland Volcanic Field are dated between 1.6 and 0.5 Ma, while the younger Auckland Volcanic Field dates back to 250 ka and is considered to be active.

Typically the lavas of the South Auckland Volcanic Field are basanite and hawaiite with local variability, and preserved lava flows are vesicular and porphyritic. Lavas of the Auckland Volcanic Field are predominantly fine-grained alkali basalt and basanite. Volcanoes in these fields have been described as following a typical progression from explosive phreatic and phreatomagmatic eruption sequences through Strombolian to Hawaiian styles contributing to a varied field of tuff rings and scoria cones, with lava fields and shields. More than half of Auckland's volcanoes produced a blast crater before in some cases going on to establish a scoria cone (Hayward, 2019).

2.7.2 GEOGRAPHICAL DISTRIBUTION

The South Auckland Volcanic Field covers an area of approximately 300 km^2 between Papakura, Waiuku, Pukekawa and Hunua. More than 100 volcanic centres are identified, typically located along or in close proximity to known faults, or inferred extensions of faults.

The Auckland Volcanic Field covers approximately 100 km^2 extending from Rangitoto and Pupuke in the north to Matukutururu and Ash Hill in the south, Mount Albert in the west and Pigeon Mountain in the east. Up to 53 volcanic centres have been named in the Auckland Volcanic Field.

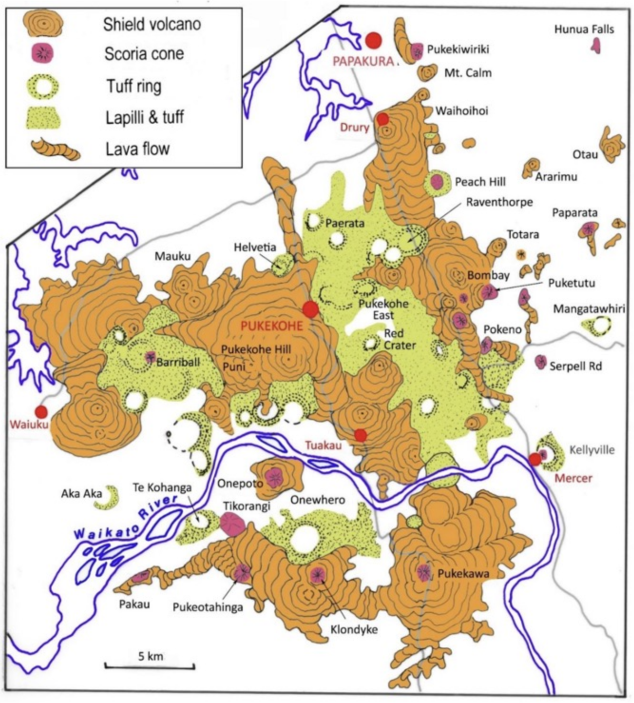


FIGURE 9: Map of the South Auckland Volcanic Field. Adapted from Briggs et al. (1994) and Nemeth et al. (2012). From Hayward (2017) Out of the Ocean into the Fire, p. 208.

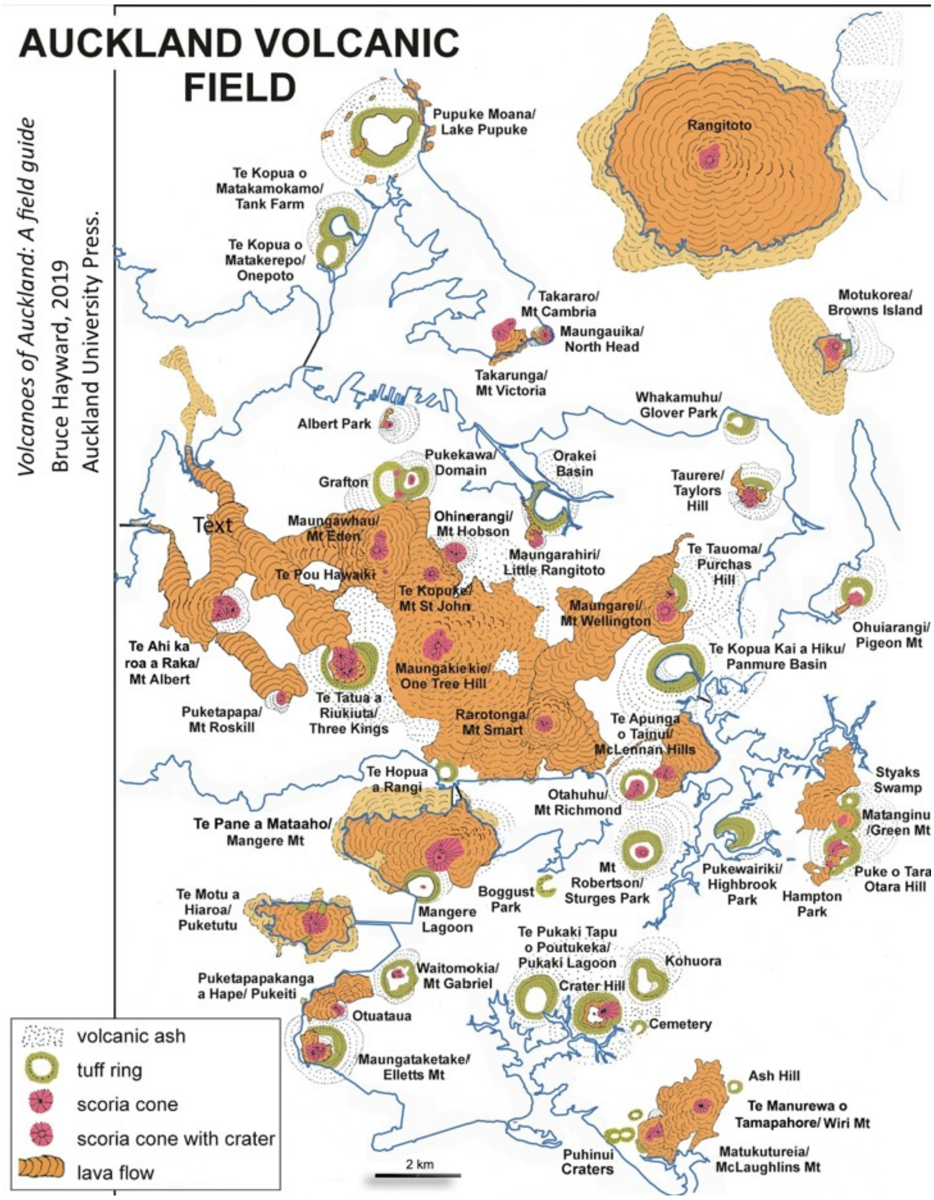


FIGURE 10: Map of the Auckland Volcanic Field from Hayward (2019) Volcanoes of Auckland: a field guide.

2.7.3 GEOMORPHOLOGY

Magmatic and effusive volcanism has produced a complex volcanic landscape comprising scoria cones, lava flows and shields, tuff rings and maars. The older volcanics of the South Auckland Volcanic Field are deeply weathered with subdued topographic relief and a substantial soil overburden, in contrast to the younger Auckland Volcanic Field which is typically unweathered. Small, steep-sided scoria cones are scattered across the field, and local sub-vertical tuff ring walls and maars reflect areas of phreatomagmatic or explosive volcanism.

Scoria cones of the South Auckland Volcanic Field are typically more rounded and subdued than their northern counterparts, reflecting their greater age. The best preserved and most prominent scoria cones are in the southern portion of the South Auckland Volcanic Field, south and east of the Waikato River. Tuff craters and maars have in many cases been infilled with younger sediments and their walls eroded by alluvial flows and anthropogenic activities.

The Auckland Volcanic Field dominates Auckland's landscape with steep-sided scoria cones towering up to 120 m over the surrounding area. Many of the smaller scoria cones have been quarried out in part or entirely over the last 100 years. Explosion craters and tuff rings have become established as lakes, tidal lagoons, and through anthropomorphic activities have been filled and drained for use as recreational space, landfills and farmland (e.g., Lake Pupuke, Mangere Lagoon). Some of the tuff rings comprise sub-vertical cliffs of welded tuff more than 10 m high (e.g., Orakei Basin).

FIGURE 11: Rotational slide on the flank of Ōwairaka (Mount Albert) after heavy rainfall (2023).

Anthropogenic activities across the Auckland Volcanic Field have resulted in well-known exposures of columnar jointed basalt (e.g., Auckland Boys Grammer School sports field) and rubbly basalt (e.g., Mortimer Pass in Newmarket).

2.7.4 GROUNDWATER

Due to its rubbly and jointed nature, basalt is typically free draining. Lava flows of the Auckland Volcanic Field make effective underground aquifers, draining along the alignment of the original stream valleys towards the sea where they emerge as springs (Hayward, 2019). Locally perched water surfaces can develop where the basalt rock is massive, or where the volcanic strata is confined by less permeable cohesive soils derived from ash or the existing underlying strata. Instability in volcanic soils are often attributed to concentration of surface water flows which can develop as a result of anthropogenic changes to the landform (e.g., paved roads with poor surface drainage control).

2.8 OTHER GEOLOGY

The NZ 1:250,000 Geological Map shows less frequent, localised outcrops of other distinct geological formations present across the region. These are briefly summarised below for completeness.

2.8.1 EASTERN VOLCANICS AND HAURAKI GULF ISLANDS

Many of the islands of the Hauraki Gulf emerged from the volcanism attributed to the eastern Miocene volcanic arc, which also formed the present-day Coromandel Ranges to the south.

Great Barrier Island consists of the remnants of an extinct volcanic caldera, with sheets of intrusive andesite and dacite lavas (dikes), volcaniclastic breccias and tuff beds forming the dominant geology of the island (Kuaotunu Subgroup, Coromandel Volcanic Group). Elsewhere, younger Rhyolite and Ignimbrite (Minden Subgroup, Whitianga Group) rocks form the central highlands, with the older Waipapa Group Terrane greywacke exposed on the northern and eastern coastlines.

Little Barrier is incorporated into the Coromandel Volcanic Zone but was formed much later than Great Barrier Island. The central island is dominated by porphyritic dacite dikes and lava flows, moving to dacite breccia around the island's margins (Haowhenua Formation, Hauturu Volcanic Group)

2.8.2 TI POINT GROUP BASALT

An isolated pocket of basalt outcrops on North East Waiheke Island and at Ti Point south of Leigh with sporadic outcrops in surrounding areas. The Ti Point Group Basalts consist of basaltic lava flows and intrusives, composed primarily of olivine phenocrysts in a clinopyroxene and plagioclase groundmass (Heming, 1980).

The Ti Point Group has been dated to 6.8 to 10.5 Ma and may be considered similar to, albeit geologically older than, the basalts of the Auckland and South Auckland Volcanic Fields.

2.8.3 KAIPARA HARBOUR

The Auckland city boundary passes westward from south of Mangawhai Village on the east coast to south of Topuni located adjacent to the inner Kaipara Harbour, and incorporates the Okahukura Peninsula which includes Tapora and Wharehine and protrudes into the Kaipara Harbour.

Within the Auckland region, the early Miocene Waitākere Group includes two subgroups; those formations whose source of clastic material was the Manukau Volcano (Manukau subgroup), and those whose source of clastic material was the Kaipara and associated satellite volcanoes (Hukatere Subgroup). The former forms the present-day Waitākere ranges, and the latter the Kaipara peninsulas.

The remnants of the Ōruawharo satellite volcano outcrops on the northern edge of the Okahukura Peninsula as a 150 m thick deposit of glassy volcanic breccia (Hayward, 2017), occasionally cut by basaltic andesite dikes and rare basaltic pillow lavas (Edbrooke, 2001) (Oruawharo Hyaloclastite, Hukatere Subgroup). Moving south across the peninsula, outcrops of primary effusive volcanics terminate, and thick sequences of volcaniclastic sandstones derived from the remobilised volcanic debris on the slopes of the volcanic edifice dominate (Pakaurangi Formation, Hukatere Subgroup). This is analogous to the Nihotupu Formation further south, only less widespread.

The south and east of the Okahukura Peninsula are composed of older Waitematā sandstones, mudstones and conglomerates (Waihangaru, Matapoura and Timber Bay Formations).

2.8.4 SOUTHERN TE KUITI GROUP OUTCROPS

Between Pukekohe and the Hunua Ranges, sporadic outcrops of Eocene Waikato Coal Measures Formation and rare Mangakotuku Formation can be observed lying unconformably above the Waipapa Group Greywacke basement. These formations comprise terrestrial carbonaceous mudstone, coal measures and rare sandstones and conglomerates (Waikato Coal Measures) and marine massive siltstone and mudstone, and glauconitic muddy sandstone (Mangakotuku Formation).

3 SLOPE INSTABILITY IN AUCKLAND

3.1 GENERAL

A comprehensive discussion of slope movement types and processes is given in Part 2 of Unit 1 and Part 2 of Unit 2. This section does not intend to replicate that information, instead presenting slope movement types most typical to the varied geological units of the Auckland region. Practitioners should not limit themselves to the guidance in the following sections, and are encouraged to employ the approaches described in Unit 2 to inform their identification and investigation of these hazards.

Slope failures encountered in the various geological formations across the Auckland region share many similarities in both failure mode and triggering mechanism.

The residual weathering of Auckland rocks typically forms a clay soil mantle less than 10 m thick, extending to depths of 25 m or more in some cases. These surficial clay layers are susceptible to, among other things, changes in pore water pressures, landform modifications, toe erosion, surcharge loading and ground shaking. Negative pore water pressures also contribute significantly to their stability, which can be reduced seasonally as wetting fronts move down through the soil.

Planar sheared surfaces and clay seams present within the residual soils frequently form the basal rupture surface of translational landslides when daylighted by excavations, unloaded by earthworks, subjected to excessive loading by placement of fill, structures or temporary loads, or by rapid pore water pressure increases.

Coarser grained soils (e.g. West Coast dune sands) are more susceptible to surface water flows, mobilising soil particles by surficial erosion leading to oversteepening and rotational landslides and debris flows.

Severe storms can cause debris flows in the steep hill country across all soil types. These consist both of flows confined by steep channels and gullies (debris flows) and those unconfined across open slopes (debris avalanches).

Many landslides that initiate as a rotational or translational slide transition to a debris avalanche or earthflow mechanism during the later stages of failure.

Rockfalls and topples occur in exposed cliff faces, bluffs and steep road cuttings, particularly within the closely fractured units (e.g. Waipapa greywackes, Northland Allochthon) and stronger volcanic formations (e.g., Waitākere Group, Auckland and South Auckland Volcanic Field basalts). Planar and wedge translational failures are less common but do occur where the rock mass structure controls the behaviour of the material. Highly to completely weathered East Coast Bays Formation rock can fail as a rotational slide, with the basal failure surface geometry influenced by rock mass anisotropy.

Slope failures induced by seismic shaking are rare in Auckland but cannot be overlooked given the regional tectonic setting of the wider North Island and proximity to active faults.

Soil creep affects many of the steep hill slopes across the region, with deformed trees, offset fence lines and slope terracettes indicating long-term soil movement.

Table 4 provides a summary of the typical modes and causes of failure encountered in each of the Auckland geological terranes and geomorphic regions. It is intended to indicate those modes and causes that most impact each group and is not an exhaustive list of all possible types of failure.

Table 4: Summary of Common Modes and Cause of Slope Failures in Auckland Geology

Auckland geology	Associated Slope Instability		Common causes of failure
	Soil	Rock	
Waipapa Group Terrane	Translational slides Rotational slides Soil creep	Falls and Topples Wedge failure Translational slides	Soil Elevated pore water pressures in the near surface soils and on basal discontinuities Rock Surficial weathering Erosion of discontinuity infills (cliffs and steep outcrops)
Northland Allochthon	Translational slides Soil creep	Translational slides	Soil Elevated pore water pressures within the residual soils, on the basal soil rock interface and the contact between the surficial residual soils and the 'transitionally' weathered soil layer Increases in moisture content causing expansion of reactive clay minerals in planar clay infilled surfaces. Exposure of the transitionally weathered zone in excavations causing translation sliding on this interface Loss of toe support (by natural or anthropogenic sources) Rock Elevated pore water pressures in highly sheared low strength rock causing deep seated planar slides
Waitematā Group	Translational slides Rotational slides Debris flows Soil creep	Falls Topples Translational slides Rotational slides	Soil Elevated pore water pressures within the residual soils and on basal discontinuities Loss of toe support (by natural or anthropogenic sources) Daylighting of sheared surfaces and clay seams in excavations Unloading causing removal of confining stress on planar surfaces and long-term pore pressure equilibration. Rock Deep-seated parallel-bedded clay seams activated through loss of toe support. Chemical weathering of rock mass defects and root infiltration leading to jacking and defect degradation. Seasonal wetting and drying cycles caused by groundwater fluctuations reducing rock mass strength in cliff faces. Cliff erosion, particularly in well-jointed rock mass.
Waitākere Group	Translational slides (Nihotupu Formation) Rotational slides (steep hill country and coastal areas) Debris flows Soil creep	Topples and Falls (Piha, Lone Kauri, Waiatarua)	Soil Elevated pore water pressures within the residual soils and on basal discontinuities Loss of toe support (by natural or anthropogenic sources) Surficial saturation on steeply formed hill slopes Rock Surficial weathering of exposed cliffs, bluffs and rock cuttings, particularly in breccias and conglomerates where sandy matrix is less resistant to weathering.
West Coast Dunes (Awhitu and Kariotahi Groups)	Rotational slides Translational slides Debris flows Soil creep		Concentrated overland flow causing scour and oversteepening Elevated pore water pressures on basal contact with older geological formations or lower permeability layers

Auckland geology	Susceptible Slope Movement		Common causes of failure
	Soil	Rock	
Takaanini Formation (Southern Lowlands)	Rotational slides Translational slides Soil creep	Rock falls and topples	Soil Elevated pore water pressures in the near surface soils and/or at contacts between layers and on basal unconformities Loss of toe support (by natural or anthropogenic sources) Rock Weathering of soft exposed rock cliffs
Auckland and South Auckland Volcanic fields	Rotational slides Translational slides Soil creep	Rock falls and topples	Soil Elevated pore water pressures within the near surface soils and on basal contact with underlying rock or Takaanini Formation Rock Excavation and blasting Surficial weathering of exposed cliffs, bluffs and rock cuttings

3.2 TRANSLATIONAL SLIDES

Translational sliding of residual soils on low strength planar surfaces is a dominant failure mechanism across most of the geological units in Auckland. The basal failure surface can be one or a combination of the following:

- Stratigraphic contacts between geological units (or individual layers within those units) having different geological properties.
- The contact between colluvium and the underlying intact ground.
- The contact between fill and the underlying original ground surface.
- Existing structure or fabric within the soil and/or rock mass comprising intersecting defects, bedding, sheared zones and crushed zones. This may include existing failure surfaces of existing landslides.

3.2.1 STRATIGRAPHIC CONTACTS

The most common form of translational sliding in Auckland geology are on bedding contacts within the residually weathered soil layer, or stratigraphic contacts within/between soil units, and at the contact with the underlying rock mass. This is demonstrated by the well-known Kepa Road Landslide, a dramatic planar slide of lithified tuff overlying residually weathered clay of the East Coast Bays Formation (Figure 12, and refer to Section 5.1 for more detail).

Regardless of the geological unit in question, the contact between two units of different permeability tends to develop a preferential water flow path or a softened zone associated with elevated pore water pressure at that interface. As a result, the effective stress of the affected soil layer is reduced and the

FIGURE 12: Kepa Road landslide. See Section 5.1 for case study.

likelihood of a landslide initiating at that contact is increased. In many cases a secondary trigger such as a change in slope loading or removal of toe support is necessary to initiate failure, however in extreme or prolonged rainfall events pore water pressures can rise to the point of initiating failure in isolation.

3.2.2 PRE-SHEARED SURFACES

Once landsliding initiates, if sufficient displacements are encountered (which published laboratory testing records indicates is generally between 100 and 500 mm in clay soils, e.g., Skempton, 1985), many Auckland residual soils exhibit strain-softening behaviour caused by the alignment of platy clay minerals on the rupture surface.

These surfaces, now at residual strength, remain in place essentially in perpetuity, and although some recovery of shear strength occurs post-failure, research has shown that only very small displacements are required for the loss of the recovered strength, and return to the residual strength (Bhat et al., 2014, Stark and Hussain, 2010).

Because of this, reactivation of ancient landslides is a major hazard in Auckland, particularly in the Miocene flysch sequences, where only minor changes in site conditions can often recommence movement on the low-strength surfaces.

Adverse impacts can be particularly pronounced when pre-sheared surfaces remain unidentified on development projects that incrementally reduce the factor of safety against instability (e.g., by unloading of the slope, construction of retaining walls etc.). Further changes in site conditions post construction (e.g., heavy rainfall) can then tip the balance from marginally stable, to entering a state of failure.

Identification of pre-sheared surfaces early in a project is therefore important for mitigation of this risk. Detailed geomorphic mapping followed by a targeted, appropriate site investigation is the best method in which this can be achieved. Further guidance regarding selection of appropriate site investigation techniques is provided in section 4.1, and more generally in Unit 2.

Pre-sheared surfaces can also be derived from tectonic movements (faulting and folding) and from progressive failure, described by Bjerrum, 1967, as the creation of a sheared surface following the redistribution of stresses in over consolidated clays post-unloading (e.g. via erosion, glacial retreat, earthworks, landsliding etc.).

Figures 13 and 14, show examples of a pre-sheared surface and features of translational landslides attributed to them.

FIGURE 13: Shear plane at the contact between residual silt-dominant (upper) and clay-dominant (lower) soil layers of the East Coast Bays Formation .

FIGURE 14: Slice through a translational relict landslide in East Coast Bays Formation Residual Soils, Orewa. Note landslide morphology features (A&B horst and graben structures, C- displaced blocks, D- increased downslope deformation and E- a basal shear surface separating lower undisturbed soil and overlying deformed, displaced soil.) The depth from ground level to the failure plane was observed to be 6 m.

FIGURE 15: Pre-sheared surface and soil fabric in Northland Allochthon

The complex tectonic history of the Northland Allochthon mean that these rocks are characterised by an abundance of tectonically derived pre-sheared surfaces. Due to the intense folding, however, they are rarely orientated in a uniform direction or sufficiently laterally continuous to give rise to large scale block type failures. This is nonetheless a common mode of failure in Northland Allochthon, but one more likely attributed to relict landslide surfaces or sliding on the soil-rock interface, like the one shown in Figure 15.

3.2.3 PARALLEL BEDDED CLAY SEAMS

Flexural slip (i.e. the sliding of individual beds in an interbedded geological sequence over one another) in response to compressive or extensional tectonic stresses has provided a mechanism for the formation of bedding parallel clay seams (Williams et. al, 2004). These horizons are reasonably common in the Miocene interbedded sedimentary formations (e.g. East Coast Bays, Pakiri, Nihotupu Formations) and Northland Allochthon, with frequency increasing in proximity to faults.

Work on a water supply tunnel in the Southern Landslide Zone demonstrated the potential for widespread continuity of the clay seams (Wyllie, 1989). Assessing the continuity and nature of such clay seams is of critical importance to developing a robust engineering geological model (Prebble, 2001).

Landform modifications such as earthworks, road cuttings, retaining wall excavations and drainage works often trigger translational landslides when these seams are daylighted in excavations, even when excavation faces are cut at shallow angles (i.e. <1V:5H) or shallow bedding dips are observed to be favourable (i.e. into the slope). Once initiated, failure rates are often slow, occurring at tens of millimetres to metres per day. Some examples of parallel bedded clay seams and associated slope failures are shown in Figures 16 through 18.

FIGURE 16: Parallel bedded clay seam in residually weathered East Coast Bays Formation sliding into a shear key excavation at a rate of approximately 20 mm/day.

FIGURE 17: 10 mm wide parallel bedded clay seam comprising highly saturated light grey clay in East Coast Bays Formation rock excavation. Moisture content of the clay seam was measured at 62.2%. Ring shear tests carried out on the clay infill recorded effective residual friction angles, ϕ'_r, of 10° and 19° (c'_r = 0 kPa).

FIGURE 18: Deep seated block slide in the Southern Landslide Zone.

FIGURE 19: Left: Rotational slide in Holocene Undifferentiated Takaanini Formation strata adjacent to a stream (left) due to ongoing toe softening and erosion
Right: Rotational Slide in Piha Formation breccia, following intense rainfall, Bethells Beach

3.3 ROTATIONAL SLIDES

Due to the deep soil profile across much of Auckland, rotational slides are common. They are mostly triggered by saturation of surficial soil layers by rainfall but are also by oversteepening of cut faces, toe erosion (as shown in Figure 19, left) or by long term equilibration of pore water pressure in cut slopes.

They are common in the recent (i.e. Pleistocene to present) sedimentary soils, or in homogeneous fill slopes, where planar defects or discontinuities which would otherwise give rise to translational type slides are sparse or absent.

Rotational slides in residual soil slopes tend to be shallow, and occur in response to heavy rainfall saturating surficial layers, particularly in high steep hill country (example shown in Figure 19, right).

3.4 ROCKFALL AND TOPPLES

Rockfall is widespread across the Auckland region, most notably in coastal settings where cliffs are formed in Waitematā Group, Waitākere Group and Waipapa Group rock strata. Rockfall hazards are also associated with exposed rock faces such as existing bluffs or head scarps, cut faces, and loose debris associated with previous landslides.

Steep cliffs on the eastern coastline of Auckland are excellent examples of the bedded nature of the Waitematā Group, with locally faulted and folded strata demonstrating a significant range in bedding orientation and continuity over short distances. Rockfall from these cliffs is a result of mechanical and chemical weathering of the rock layers, with blocks ranging in size from millimetres to metres (Figure 20). The size and frequency of the rocks released is governed by the rock mass structure, influenced by the effects of vegetation at the surface and its associated root structures infiltrating the defect openings.

Rockfall is also a common occurrence on Auckland's west coast cliffs formed in the Waitākere Group geology, particularly in the Piha Formation breccias and conglomerates (Figure 20), where the lower resistance to weathering of the sandy matrix can result in dislodgement of basalt and andesite boulders.

Rock fall from cut slopes formed in jointed basalt and tuff units of the Auckland and South Auckland Volcanic Fields is also common. The basalt rock mass is typically very strong, with closely to widely spaced joints and a variable weathering pattern. Toppling of columnar jointed basalt rock can occur in excavations, with spalling of the less welded tuff and highly weathered basalt units resulting in episodic rock fall.

FIGURE 20:
Top Left: Vegetated East Coast Bays Formation coastal cliff with accumulated rock fall debris at the toe on the eroded shore platform.
Top Right: Rock topple in an East Coast Bays Formation coastal cliff, block dimensions 3 m by 1 m. Takaanini Formation alluvium forms the upper 1 to 2 m of the cliff profile.
Bottom Left: Rock fall in Piha Formation cliffs at Piha.
Bottom Right: 0.6 m diameter Waitarua Formation rock released from a rock outcrop above Piha.

3.5 DEBRIS FLOW AND DEBRIS AVALANCHE

Debris flows and avalanches are common in the Auckland region, particularly in areas characterised by deeply weathered granular soils and highly fractured rock masses. Rock avalanches are typically observed in areas underlain by Waipapa Group and Waitākere Group rocks where the rock masses are sheared and shattered by ancient faulting.

The west coast Awhitu and Kariotahi Group dune sequences are particularly susceptible to rapid increases in water content and concentrated overland flow, which during periods of intense rainfall regularly trigger widespread debris flows like the one shown in Figure 21. The free draining nature of the soils also allows water to infiltrate the soil mass until it encounters relatively impermeable strata, leading to elevated pore water pressures at the stratigraphic contact. This can initiate translational slides that transition to debris flows as the soil becomes fully saturated.

This transition from slide to flow movement also affects clay slopes, where the initial onset of failure during rainfall opens up cracks and fissures, facilitating increase in soil moisture content. More displacement leads to higher moisture contents until the soil essentially loses its shear resistance and rapid flow movement initiates. This can result in significant volume flows, like the one in Figure 22.

FIGURE 21: Debris flow in Kariotahi Group sand dunes (aerial view of Figure 7).

FIGURE 22: Rotational slide transitioning to debris flow in East Coast Bays Formation resulting in inundation of the road and railway line, with debris reaching the stream.

FIGURE 23: Debris avalanche in Pakiri Formation residual soil overlying rock. Saturated soil and entrained vegetation flowed over the rock shelf to inundate the railway line.

FIGURE 24: Debris avalanche in Auckland Volcanic Field sandy tuff soil.

3.6 SOIL CREEP

Creep is defined by United States Geological Survey (USGS) as '*the imperceptibly slow, steady, downward movement of slope-forming soil or rock. Movement is caused by shear stress sufficient to produce permanent deformation, but too small to produce shear failure.*'

The rate of movement is influenced by precipitation, clay content, clay mineralogy, temperature ranges, geology, slope angle, slope aspect and vegetation coverage.

There are several known mechanisms by which creep movement occurs in Auckland slopes:

1) Downslope viscous flow.
2) Seasonal wetting and drying of surficial soils causing expansion and shrinkage of clay minerals resulting in a net downslope movement on slopes.
3) Wedges of soil forming terracettes via gradual downslope movement on a basal surface (Figure 25).

Evidence includes deformed or leaning trees (Figure 26), fence posts or telegraph poles, and sets of slope parallel terracettes in steep hill country (Figure 27).

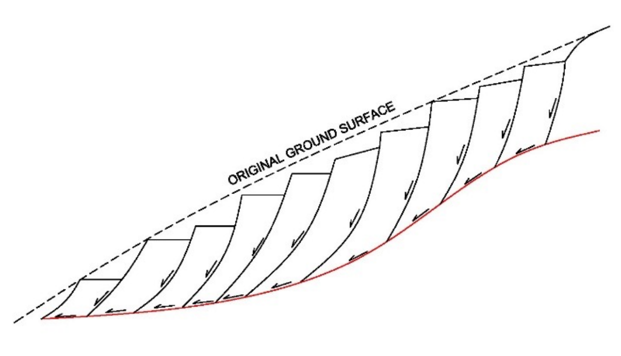


FIGURE 25: One mechanism for terracette formation on steep slopes

Shallow soil creep is broadly accepted as operating on natural slopes in Auckland at or steeper than 1V:4H, or 14°. Soil creep does also occur on cut and fill slopes at these angles, but can be minimised through engineering design (e.g., inclusion of geogrid reinforcement, geocell face stabilisation, etc.).

FIGURE 26: Rotation of juvenile tree trunks indicative of shallow-seated soil creep over the duration of the life of the tree.

FIGURE 27: Terracettes across a soil slope indicative of soil creep.

4 INVESTIGATIONS, ANALYSIS AND DESIGN

4.1 SUITABLE GEOTECHNICAL INVESTIGATIONS

Any land instability investigation should begin with a comprehensive desktop study and geomorphological assessment to establish a sound understanding of the geological setting and history of instability. This then forms the basis from which the subsurface investigation can be designed. A full discussion on surface investigation methods is given in Unit 2.

Subsurface investigations for the purposes of assessing slope stability should first consider what modes of failure are likely to occur. Practitioners can then select investigative techniques that allow them to identify or assess features that are critical in enabling the development of a suitable geotechnical model.

As described in Section 3, the dominant mode of failure across much of the Auckland region is translational sliding on some form of basal contact (e.g. the soil-rock interface, clay seams, pre-sheared surfaces). It is important therefore, that investigation techniques are selected that enable the observation of these surfaces.

The investigative techniques described in Parts 2 and 3 of Unit 2 are appropriate and applicable to Auckland's complex geological setting, however some subsurface techniques yield more valuable data than others in specific areas. The following sections provide specific recommendations for practitioners undertaking subsurface investigations in certain geological settings where collective experience has demonstrated their effectiveness.

The Northland Allochthon is addressed in detail in Unit 7A.1 and the reader is directed to that document for a full discussion on suitable investigation techniques. However, the following techniques are advised as a baseline when undertaking a slope stability assessment in these materials.

Table 5: Suitable Investigation Techniques for Waipapa Group

Investigation Technique	Comment on Applicability
Cored machine borehole (with in-situ testing)	Allows for inspection of bedding and defects for evidence of pre-sheared surfaces, and observation of defect spacing for rock mass rating.
Test Pits	Particularly effective in Waipapa Group terrain due to the relatively thin residual soil layer (typically <5m) allowing for examination of the soil/rock contact and some data collection on rock mass structure (excavator can often penetrate the highly fractured weathered rock). In landslides, allows for observation and measurement of shear planes and in some cases, sample collection for laboratory testing.

Table 6: Suitable Investigation Techniques for Northland Allochthon

Investigation Technique	Comment on Applicability
Cored machine borehole (with in-situ testing)	Allows for inspection of weathering, lithologic descriptions and relationships including stratigraphic contacts, descriptions of tectonic fabric and overprinting including shearing, brecciation and fault gouge. Allows for insitu testing to establish soil and rock parameters e.g. pressuremeter testing, and the installation of monitoring instrumentation for groundwater monitoring and inclinometers.
Cone Penetration Test (CPT) and Dilatometer Test (DMT/SDMT)	Allows for identification and development of soil parameters for the upper weathered profile.
Test pits or trenches	In the pervasively sheared Mangakahia Complex units, the residually weathered soil layer is often less than 5 m thick. Test pits are essential for allowing inspection and description of the soil fabric and rock contact.

Table 7: Suitable Investigation Techniques for Waitematā Group

Investigation Technique	Comment on Applicability
Cored machine borehole (with in-situ testing)	Allows for inspection of bedding and stratigraphic contacts for evidence of pre-sheared surfaces or clay seams, although can be unreliable for detailed defect descriptions. Allows for installation of monitoring instrumentation (e.g., inclinometers, piezometers) for measurement of slow moving or creeping landslides.
Test pits or trenches	Excavations targeting shallow landslides (< 5 m), or at the crown and toe of deeper-seated landslides, allow for inspection of failure surfaces over a greater length to collect detailed information on their orientation, shape and coating, as well as details of bedding and defects. Where excavations are safely benched or shored for entry, undisturbed samples of shear planes may be possible to collect using a driven sampler tube for laboratory testing. Allows identification of in-situ (i.e. in-tact, structured, undisturbed) soil and displaced (i.e. deformed, non-intact, disturbed) soil.
Hand Auger Boreholes	Can aid in the development of ground models on undulating sites where access for machines may be problematic. Effective in identifying depth to important horizons in the soil (e.g. depth to hard interface, depth of landslide debris) and depth of perched groundwater in the deep marine turbidite units. Less effective in the marine channel conglomerates due to difficulties penetrating gravels. Data they provide should be considered of limited accuracy and reliability, and limitations in their maximum penetration depth (usually 5m) and inability to penetrate hard soil layers and rock mean they are not appropriate on their own to form a complete assessment of the risks on Waitematā Group slopes. Hand augers should therefore only be used for the purposes of slope stability assessment in conjunction with other more robust investigation techniques (e.g. machine boreholes).

In addition to these routine techniques, observation of bedding parallel clay seams in-situ within the East Coast Bays Formation rock can be possible when employing inspection shafts. In the work reported on by Williams and Prebble (2014), the use of inspection shafts provided continuous exposures and allowed for direct observation of the soil and rock profile as well as clay seams and defects. It is possible to collect undisturbed samples using a driven sampler tube for laboratory testing.

Inspection shafts are rarely undertaken in routine geotechnical projects, however in large-scale infrastructure projects where parallel-bedded clay seams are of particular concern their use could be considered.

Table 8: Suitable Investigation Techniques for Waitākere Group

Investigation Technique	Comment on Applicability
Cored machine borehole (with in-situ testing)	Gravel to boulder sized clasts in the lavas and volcanic conglomerates and breccias of the Waitākere Group soils can be hard to penetrate with CPT's or hand augers. Machine boreholes are effective in penetrating the soil/rock interface, assessing the depth of weathering of the rock mass and observation of surfaces, particularly in Nihotupu and Piha Formation.
Test Pits	Excavations targeting shallow landslides (< 5 m), or at the crown and toe of deeper-seated landslides, allow for inspection of failure surfaces over a greater length to collect detailed information on their orientation, shape and coating, as well as details of bedding and defects. Where excavations are safely benched or shored for entry, undisturbed samples of shear planes may be possible to collect using a driven sampler tube for laboratory testing.
Hand Augers	May be difficult to penetrate to maximum depth due to gravel inclusions in some lithologies, but effective in Nihotupu Formation and to a lesser extent Piha Formation in gathering information for construction of ground models.

Table 9: Suitable Investigation Techniques for Awhitu / Kariotahi Group

Investigation Technique	Comment on Applicability
Cored Machine Boreholes	Enables the observation of layers which may form the basal surface of translational slides (e.g. basal contact with Miocene lithologies). Allows for installation of piezometers at depth to observe groundwater table.
Cone Penetration Test (CPT)	The degree of cementation in Awhitu and Kariotahi Group Soils has a large impact on the stability characteristics of these slopes. Methods of interpretation of CPT and sCPT data allow for the assessment of the degree of cementation and structure. However high levels of cementation, or narrow harder bands may inhibit the penetration of the CPT to the required depth.
Hand Augers / DCP	Of limited value but can be used to identify the depth of the surficial fine-grained layer. DCP/Scala testing can indicate the density of the near surface sand layers.

Table 10: Suitable Investigation Techniques for Takaanini Formation

Investigation Technique	Comment on Applicability
Cored machine borehole (with in-situ testing)	Allows for correlation of CPT tests and in situ SPT (and in some cases shear vane) testing. Permits collection of undisturbed soil samples for laboratory testing establish shear strength parameters.
Cone Penetration Test (CPT)	Recommended in highly heterogeneous soils for identifying thin layers of sensitive soils (e.g., pumiceous silt and sand layers) that warrant more careful consideration in the geological model. Can be useful in estimating the shear strength of clay soil layers, however this is most effective when correlated to in situ shear vane tests and/or laboratory testing. Even very thin pumiceous silt layers have been known to prevent the penetration of the CPT to the required depth, however the identification of hard layers within the soil profile is still of benefit to the slope assessment.
Hand Auger Boreholes	Useful for obtaining in situ soil undrained shear strength profiles using a shear vane. Beneficial in conjunction with CPT testing to verify the inferred shallow ground profile (typically less than 5 m).

Table 11: Suitable Investigation Techniques for Auckland Volcanics

Investigation Technique	Comment on Applicability
Cored machine borehole (with in-situ testing)	Machine boreholes may be required where confirmation of the nature of the underlying basalt is required to inform a suitable engineering geological model, including retrieval of core samples for laboratory strength testing. Cored boreholes in Auckland Volcanics can be slow and costly due to slow drilling rates in basalt rock, and in cases where fractured rock and gravel becomes lodged in the core barrel prohibiting core recovery.
Test Pits	Where machinery can access the test location, test pits provide an effective approach for collection of data from the residually weathered ash and tuff soils (where present), and can penetrate to some extent into the underlying rubbly basalt. Progress will be governed in most cases by the capabilities of the excavator.
Hand Auger Boreholes	Useful to assess the thickness and undrained shear strength of cohesive volcanic soils using a shear vane, and density of granular layers using a Scala penetrometer. Advancement of the hand auger can be limited by gravel inclusions in the soil or when approaching the rubbly top of flow.
Percussion drilling	Generally not useful in slope stability analysis, but can be beneficial when confirmation of the general nature and thickness of the basalt rock is needed to inform analysis and design without core recovery or detailed soil and rock logging.

4.2 GEOTECHNICAL PARAMETERS

The dominant failure mechanism on many unstable slopes across Auckland is translational sliding of the upper fine grained soil mantle over more competent, underlying low permeability layers, or on a basal pre-sheared surface or clay seam within the soil or rock mass.

The parameters of this surface or layer often drives the geotechnical design of landslide susceptible sites in Auckland, with the assumption of residual strength parameters considered appropriate.

The presence of these surfaces can be difficult to identify in site investigations due to their sparsity and often inconspicuous appearance. It is important that suitable investigative methods are selected to enable the observation of defects, particularly in the lower portions of the soil profile and top of the weathered rock. Nonetheless, positive identification may in some cases be unattainable, and where surface geomorphology indicates the presence of recent or relict instability, it is considered good industry practice to infer the presence of failure planes on an interface that regularly forms a basal rupture surface in the relevant geology (e.g. the base of the soil mantle, rock interface).

In the case of rock slopes, slope instability in very weak rock masses (e.g., highly to completely weathered East Coast Bays Formation) may be assessed as a hard soil slope using a circular failure mechanism, and in that case the use of soil parameters would be appropriate. Analysis of true rock slopes requires an understanding of the rock mass strength, which requires both intact rock strength parameters and discontinuity data. Unit 3 presents robust guidance on the assessment of rock slopes (CSections 3 through 8).

Table 12 provides an overview of soil parameters for each of the discussed geological units and geomorphic regions, with the exception of Northland Allochthon units which are intended to be dealt with separately in Unit 7A.1. In all cases, back analysis may be carried out in accordance with the guidance given in Unit 3 for failed and intact slopes. Considerable discussion on suitable methods for the evaluation of soil shear strength is also given in Unit 3.

If the consequence of slope movement is minor or low (as per Unit 3, Table 8) then the parameters listed in Table 12 may be used as a guide to inform parameter selection in conjunction with good engineering judgement. In all other cases, justification of the soil parameters used must be made with reference to site specific testing, using the methods described in Table 12, and with the parameters listed in Table 12 used only as a guide.

The ACCoP provides information on the selection of suitable parameters based on the availability of laboratory and in-situ testing data, and the sensitivity of a design to a given parameter (Section 2.6.2). This should be referred to prior to commencement of geotechnical designs in the Auckland region.

It is important for practitioners to consider that adoption of generalised parameters (i.e., non-site specific) is usually conservative and may result in conservative designs. The cost of this approach may outweigh the cost of a targeted, robust suite of representative soil tests, allowing for the adoption of more realistic parameters.

Well documented empirical relationships exist between various soil properties (e.g. PI, LL, M) and strength parameters (e.g. φ', S_u). If these relationships are utilised, it is important that the practitioner understands the data which underpins the empirical relationship, as they may be inappropriate in certain applications. It is desirable for priority to be given to the direct measurement of a parameter via specific testing, as opposed to reliance on empirical relationships.

Table 12: Indicative Geotechnical Design Parameters for Soil Slopes

Applicable Groups	Parameter	Typical Ranges of Parameter	Common Methods to Derive Parameter[1]	Comments
Waitematā Group, Waitākere Group, Waipapa Group	Residual effective stress strength parameters of failure plane / clay seam	$c'_r = 0$ kPa $\varnothing'_r = 8°$ to 20°	Back Analysis Ring Shear (ISO 17892-10:2018 Part 10) Atterberg Limits, Hydrometer	If ‘waviness’ of the failure surface can be demonstrated, elevated shear strength parameters can be adopted (Williams et. al, 2004), however this is difficult to achieve in practice, and it is therefore advisable for residual strength parameters to be inferred. Empirical relationships with PI, LL and clay content possible.
	Peak effective Stress Strength Parameters of Residual Soil	c’ = 2 to 10 kPa ∅’ = 28° to 32°	Back Analysis CU / CD Triaxial CPT and DMT	CPT and DMT can be correlated to effective stress shear strength (Mayne, 2016).
West Coast Dunes	Peak effective stress strength parameters of dune sands	c’ = 0 kPa* ∅’ = 32° to 45°	Back Analysis CPT/sCPT, DMT, SPT Direct Shear Testing Particle Size Distribution	CPT/sCPT can determine the extent of cementation (in accordance with Schnieder & Moss, 2011) to allow use of higher strength parameters. Direct Shear Testing will provide lower bound, worst-case parameters due to the destruction of cementation during sampling. Triaxial testing may enable accurate parameter selection if cementation can be preserved during sampling. *Cohesion may be adopted if cementation is observed or demonstrated by back analysis.
Northland Allochthon	Refer to Unit 7A.1			
Takaanini Formation	Effective stress strength parameters of inorganic soil	c’ = 0 to 2 kPa ∅’ = 22° to 28°	Back Analysis CU / CD Triaxial CPT and DMT SPT (granular soils) Atterberg Limits, PSD and Hydrometer	Normally consolidated to slightly over consolidated sedimentary clays should comprise a drained cohesion (c’) of 0 kPa. Empirical relationships with PI, LL and clay content possible for clay soils, and particle size distribution for granular soils. Organic soils and peat require careful consideration and should be supported by back analysis and testing.
Auckland and South Auckland Volcanic Fields	Effective stress strength parameters for residually weathered cohesive soils	c’ = 2 to 4 kPa ∅’ = 28° to 32°	Back Analysis CU / CD Triaxial	Collection of undisturbed samples in non-welded tuff can be challenging, particularly in granular soils.
	Effective stress strength parameters of non-welded tuff	c’ = 0 to 1 kPa ∅’ = 30° to 35°	Back Analysis CU / CD Triaxial Particle Size Distribution (granular soil)	

[1] For descriptions of these tests refer to Unit 3

4.3 METHODS OF ANALYSIS

A detailed breakdown of the underlying principles of widely used analysis methods discussed in this chapter is provided in Unit 3.

The following section sets out specific techniques which are advantageous in the modelling of Auckland specific terranes.

4.3.1 QUALITATIVE ASSESSMENT

Given the inherent uncertainties associated with modelling the ground, particularly in residual soil slopes (e.g. Waitematā Group, Waitākere Group, Waipapa Group, Northland Allochthon), qualitative assessment methods should be considered equally as important as quantitative methods. Wesley (2010) lists the following four essential qualitative methods:

1. **Visual observation of the slope:** In the context of Auckland slopes, visually, those affected by historic or active slope movement often comprise irregular (i.e. not smooth) surface contours, slope mounds and upper concave head scarps. Ponding water in flattened slope portions can be evidence of sag ponds above grabens, or springs emanating from low permeability discontinuities or shear zones.
2. **Geological appraisal of the slope and surrounding area:** Understanding the local geological context (discussed in Section 2), including the structural geology, can provide insights into whether a slope is likely to remain stable. Auckland specific examples of this include:
    - **Formation of steep rock cuttings:** The closely fractured nature of Waipapa Group greywackes would present a much greater rock fall hazard than the same excavation in Waiatarua Formation andesite lava flows;
    - **Formation of cut slopes:** A 30° cut slope in Awhitu Group sands would be expected to perform much more favourably than the same slope formed in highly deformed and geologically complex Northland Allochthon residual soils.
    - **Important contacts:** Understanding the depth and orientation of key horizons can allow the practitioner to conceptualise the likelihood of a translational slope failure (e.g. the contact between Auckland Volcanic Field tuff and underlying sedimentary soils, or bedding orientation within Miocene turbidite layers).
3. **Review of aerial photographs:** Local variations in ground conditions can have significant effects on stability, even in geologies which are broadly understood, and where similar slopes in proximity to a site are considered stable.
    - The inspection of aerial photography, digital elevation models and topographic maps can give good indications of the susceptibility of local slopes to landsliding. Furthermore, changes in land use may have masked the surface expression of historically unstable areas. Auckland Council's extensive collection of publicly available online aerial photographs (https://geomapspublic.aucklandcouncil.govt.nz/viewer/index.html) dating back to 1940, along with other freely available online platforms (e.g. Retrolens) are a valuable resource that should be utilised in the planning phase of projects of all scales.
4. **Observing the performance of nearby slopes in the same geology:** Observation of similar nearby natural slopes, cliff exposures and road cuttings can provide insights regarding suitable cut slope angles, bedding orientation, structural defects, material types, hydrogeology and general slope performance.

4.3.2 QUANTITATIVE ANALYSIS

It is important to approach computational slope stability analysis with a good understanding of the engineering geological model, predicted slope instability mechanism, and the critical elements to which the analysis is sensitive (e.g., groundwater, soil strength parameters, etc.). The computational analysis should be used to support the designer's understanding of the predicted performance of the slope, and where the results are unexpected, the designer should already have a good understanding of why this may be the case.

4.3.2.1 Limit Equilibrium

Standard 2D Limit Equilibrium (LEM) analysis methods are generally appropriate for analysing slopes across most Auckland terranes. Consideration should be given to the most likely failure mode when selecting calculation methods, as some methods may not be suitable (e.g. Janbu unsuitable for rotational failures, Bishop unsuitable for translational failures). Generally, the Morgenstern & Price method would be considered a reliable means of searching for both rotational and translational failure modes in Auckland soils, with the addition of other more conservative methods being relied on for sanity, peace-of-mind checks.

The search method selected for analysis should also reflect the failure mechanism for the slope being assessed, with translational slides postulated as the dominant failure mechanism operating across most of the geological units in Auckland. Optimisation of analyses often leads to significant reductions in the factor of safety and identification of complex failure shapes, and is therefore strongly recommended.

4.3.2.2 Groundwater

One of the main challenges in undertaking a slope stability analysis in Auckland residual soil slopes, is how to accurately model pore water pressures in the slope. Most landslides encountered in Auckland are triggered by heavy rainfall, indicating a clear causal relationship between pore pressure development in the soil, and the initiation of failure.

The fine-grained residual soils also tend to exhibit two piezometric surfaces; a shallow perched piezometric surface, and the deep regional groundwater table usually encountered within the underlying rock.

It is appropriate therefore, to incorporate these two elements into the slope stability analysis of Auckland residual clay slopes. This is commonly carried out by applying the pore pressure parameter Ru to the residual soil layers, initially at a low or moderate level for the normal groundwater condition and then elevated to model a worst-case groundwater scenario (i.e., a large storm event). Refer to Unit 3 for guidance on employing Ru in slope stability analysis, as it is most applicable for simple single layer models.

Alternatively, a wetting front may be assumed by reducing the drained cohesion in elevated groundwater analyses (see Unit 3), or the upper perched groundwater table may be assumed as the only piezometric surface, with everything beneath that line assumed to be fully saturated. This may, however, give over-conservative results or not fully reflect the increased risk posed by the worst-case condition (i.e., a large storm event).

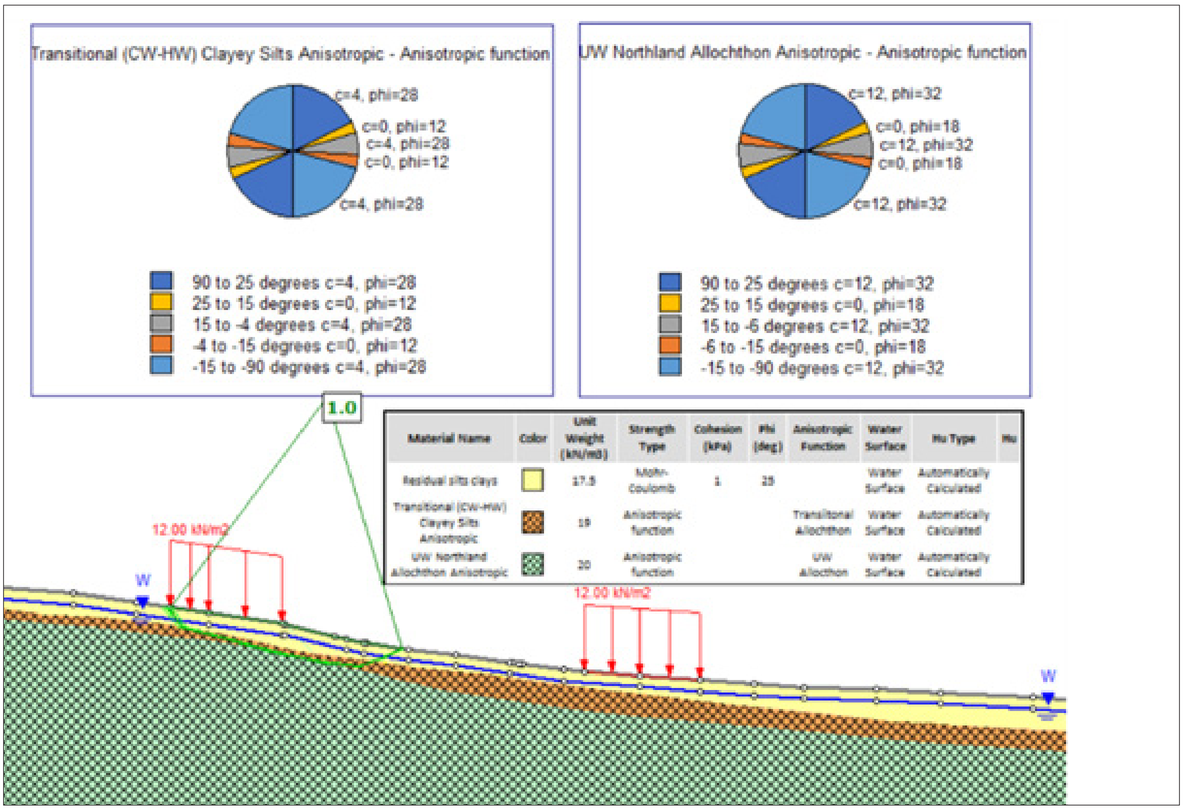
On Auckland fill sites overlying natural clay soils, it is also appropriate to assess the stability of the site when the natural soils are in the undrained condition. This can be carried out in terms of effective stress by use of excess pore water pressure functions available in most widely used LEM stability analysis software (e.g. Slide 2, Slope/W).

A rapid drawdown analysis case should be undertaken for slopes that may be susceptible to rapid reductions in adjacent water levels, such as dams and embankments, and natural slopes adjacent to water bodies that may be exposed to flood and tidal water level changes. The analysis allows for consideration of the transient seepage and porewater pressure conditions that develop in the saturated soil and remain in the short-term transient case when the adjacent water level is lowered rapidly.

4.3.3 SPECIAL CASE MATERIALS

The pervasively sheared fabric of the Northland Allochthon means that identification and selection of appropriate parameters for slope stability analysis is not straightforward, mainly due to the unreliability of laboratory testing methods in these materials. Methods that can be adopted to address this are:

- Use of back analysis of failed slopes to determine likely soil parameters for the relevant layer.
- Use of residual strength parameters.



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Anisotropic Function	Water Surface	Hu Type	Hu
Residual silts clays	🟨	17.5	Mohr-Coulomb	1	25		Water Surface	Automatically Calculated	
Transitional (CW-HW) Clayey Silts Anisotropic	🟧	19	Anisotropic function			Transitional Allochthon	Water Surface	Automatically Calculated	
UW Northland Allochthon Anisotropic	🟩	20	Anisotropic function			UW Allochon	Water Surface	Automatically Calculated	

FIGURE 28: Example of Anisotropic Strength Modelling of Northland Allochthon (Please note – parameters shown are site specific, and are not suitable to be considered generalised parameters)

- Incorporation of anisotropic analysis functions which can be used to assign residual strength parameters when the base inclination of slices is unfavourable (i.e. downslope or propagating to the ground surface at the toe), and peak strength parameters elsewhere. This incorporates the negative impact of continuous sheared surfaces in unfavourable orientations but assumes that all other randomly orientated sheared surfaces do not result in significant reductions in effective stress strength parameters for the bulk material.

Analysis and mitigation strategies in Allochthon terrain require a conservative, risk-based approach and accordingly they warrant a full and complete discussion to be included in Unit 7A.1 (in preparation).

4.3.4 DEBRIS FLOWS AND DEBRIS AVALANCHES

All steep natural soil slopes are susceptible to debris flows or debris avalanches given sufficient rises in moisture content. They most frequently occur during severe storms.

Flows cannot be accurately modelled in conventional LEM stability analysis software, so it is necessary to carry out a risk assessment, informed by the calculation of likely runout distances by analytical methods based on empirical data.

Unit 6 discusses the assessment, analysis and mitigation of debris flows in detail.

Management of the debris flow hazard should focus on reducing the likelihood and/or impact of a flow's occurrence. Locating structures, roads and services away from runout areas is recommended.

In Auckland, the flow hazard is greatest near high steep slopes, particularly in the west coast dune sequences (Awhitu and Kariotahi Groups). The risk is greatest to people and structures at the toe of these slopes.

4.3.5 ROCKFALL

The approach to rockfall analysis and design is the same in Auckland as it is for the rest of the country. However, The assessment of rockfall hazards in Auckland is less common. It's important to undertake a robust field mapping exercise to understand all of the relevant site conditions before undertaking any analysis. Practitioners should use simple 2D models to confirm their understanding before reaching for complicated 3D analysis approaches.

Part 8 of Unit 1 presents a guide to the assessment and analysis of rockfall hazards, and Unit 5 (in preparation) will build on that guidance to complement the existing MBIE guidance document (2016) for design of mitigation structures.

In the absence of seismic loads, rockfall events in isolation are rare and episodic. On natural slopes in Auckland, rockfall is often a secondary hazard where a landslide creates a head scarp bluff that forms a new rockfall source area. In these cases, the rockfall hazard may not emerge until after the initial landslide event when weathering of the source area creates the opportunity for blocks to release. In these settings, mitigation measures are likely to be designed to address multiple hazards (i.e., debris flow and rockfall).

This is obviously not the case in construction settings where excavations resulting in cut faces in rock will require temporary and permanent design solutions to mitigate the rockfall hazard, and where construction work results in vibrations that can trigger block release (e.g., scoria cone slopes in Auckland Volcanics).

4.3.6 SOIL CREEP

Current analysis of soil creep relies on judgement, proven local knowledge and accounting for creep in the design of structures or development proposals to mitigate its effects.

It is accepted that shallow soil creep, to some extent, will affect slopes in Auckland at or steeper than 1V:4H, or 14°. Paul et al. (2024) observed relatively large creep movements in peat slopes of gradients of <5°, indicating that practitioners should exercise caution when considering creep effects in peat soils (e.g. Ardmore Member, Takaanini Formation). The same is true for some soil units of the Northland Allochthon, which can also slide on slopes flatter than 10°.

Structures located on or near these slopes should be designed to accommodate movements attributed to creep, usually by founding the structure on piles designed to resist lateral loads.

Strategies to reduce the magnitude of creep displacements should seek to limit moisture content rises in slopes. Specific methods that can be employed to mitigate creep in Auckland are as follows:

- Formation of slopes from engineered fills instead of cuts, consisting of closely monitored testing specifications to limit moisture content and air voids.
- Installation of geogrids into fill slopes has been observed to reduce creep displacements
- Planting of slopes with vegetation reduces the moisture content of the near surface soils by limiting infiltration and removal of soil moisture via transpiration
- Installation of counterfort drains into slopes at suitable lateral spacings (See Price & Fitch, 2017 for Auckland specific guidance)
- Surface water controls (e.g. bunds or swales behind slope crests) to divert water away from slope faces
- On low density developments, avoidance of natural slopes by positioning structures on flat areas away from steeper gradients.

4.4 DESIGN CONSIDERATIONS

4.4.1 SEISMIC

The GNS active fault database depicts several active faults within the Auckland region (Figure 29). Earthquake shaking is predicted to be more severe in southern Auckland, near to the mapped active faults.

As Auckland lies in a region of low seismicity compared to the rest of the country, seismic analysis and design of slopes is often undertaken with conservatism due to the low predicted peak ground accelerations (PGA) for the region. In the ACCoP, a pseudo-static seismic load case using the ULS PGA is the suggested development scenario for a global stability analysis. Guidance on displacement based seismic design is included in Section 2.6.5 of the ACCoP and a detailed discussion is included in Unit 3.

Practitioners should refer to the Earthquake Geotechnical Engineering Modules published by NZGS and MBIE for calculation of PGA and guidance on assessing the effects of seismic shaking on slopes. More nuanced approaches for deriving seismic demand parameters are used when appropriate for the analysis, these include TS 1170.5:2025, site-specific probabilistic seismic hazard assessment (PSHA) and the recently updated National Seismic Hazard Model (NSHM, 2022).

Guidance to practitioners for undertaking seismic slope stability analysis is provided in Section 17 of Unit 3.

4.4.2 CUT SLOPE DESIGN

Due to Auckland's predominantly undulating topography, it is a routine requirement to form cut slopes across projects of various scales, from the formation of building platforms for single dwellings, to excavation of roading corridors for major highways.

Integration of Auckland specific industry knowledge in designs is useful in maintaining the stability of these slopes. The following presents some of these insights, as well as recommendations from international sources.

Natural residual soil slopes are prone to translational sliding, making the assessment of cut faces for planar defects an important step in maintaining adequate factors of safety against instability. Notably, where polished surfaces or clay seams are daylighted, cut slopes have been reported to fail at slope angles of as low as 10° due to the removal of toe support. In such cases, it is often necessary to undercut (i.e., excavate back beyond the design slope face) and reconstruct the slope with engineered fill and drainage to form a buttress against the excavated face (i.e., buttress fills), or support the excavation with a retaining wall.

FIGURE 29: Active Faults of Auckland. A: Wairoa North Fault, B: Wairoa South Fault, C: Paerata Fault, D: Pukekohe Fault, E: Aka Aka Fault

Identification of the bedding orientation and discontinuities should be considered a key design requirement; however translational cut slope failures can still occur when orientations of these are considered favourable for stability (i.e., dipping into the slope).

Practitioners should take a particularly cautious approach when proposing cut slopes, even temporary in nature, down from development boundaries. Slope failures in these circumstances are a common cause of property damage in Auckland, when the onset of instability extends beyond the site boundary. Risk reduction methods in these cases include (but are not limited to):

- Installation of subsurface drainage at suitable lateral spacings, allowing sufficient time for the drains to have a positive effect on porewater pressures and having assessed any potential drawdown effects on neighboring property, structures, roads or services, prior to excavation of cut slopes.
- Installation of temporary or permanent retention structures adjacent to site boundaries, prior to commencement of excavation works (i.e., top-down construction methodologies).

One of the primary hazards associated with cut slope construction is the long-term response of pore water pressure. Initially, water pressures tend to decrease in response to the reduction in confining stress following excavation. In the low permeability clay soils of Auckland, it may take several years for groundwater to return to the equilibrium hydrostatic or steady state condition, resulting in a delayed reduction in effective stress, and a long-term trend of swelling and softening of slope forming clays.

East (1974) presented three cut slope failures in Waitematā Group residual soil slopes that occurred 1 to 5 years after the cut slopes were formed. These failures were caused in part, by delayed equilibration of pore water pressure. The paper and its findings are discussed in detail in Section 5.4, and provide valuable insights into risks associated in construction of cut slopes in these soils.

FIGURE 30: Temporary cut slope in a semi-intact block of residually weathered Pakiri Formation soil. Groundwater seepage is encountered at the toe in a silt-dominant soil layer, at or close to the basal failure surface of the block.

In reality, most Auckland residual soil slopes are only partially saturated, and due to their abundant structural defects, fissures, bedding and interbedded stratigraphy (i.e., alternating high and low permeability layers) their bulk permeabilities are likely higher than fine-grained sedimentary soils (Wesley, 2010). Pore pressures may therefore respond relatively quickly, particularly where groundwater tables are encountered at depth and where the soils are relatively free draining.

This will not be the case at all locations, so it is important that groundwater monitoring is carried out to understand the prevailing hydrogeological conditions at a site prior to formation of cut slopes.

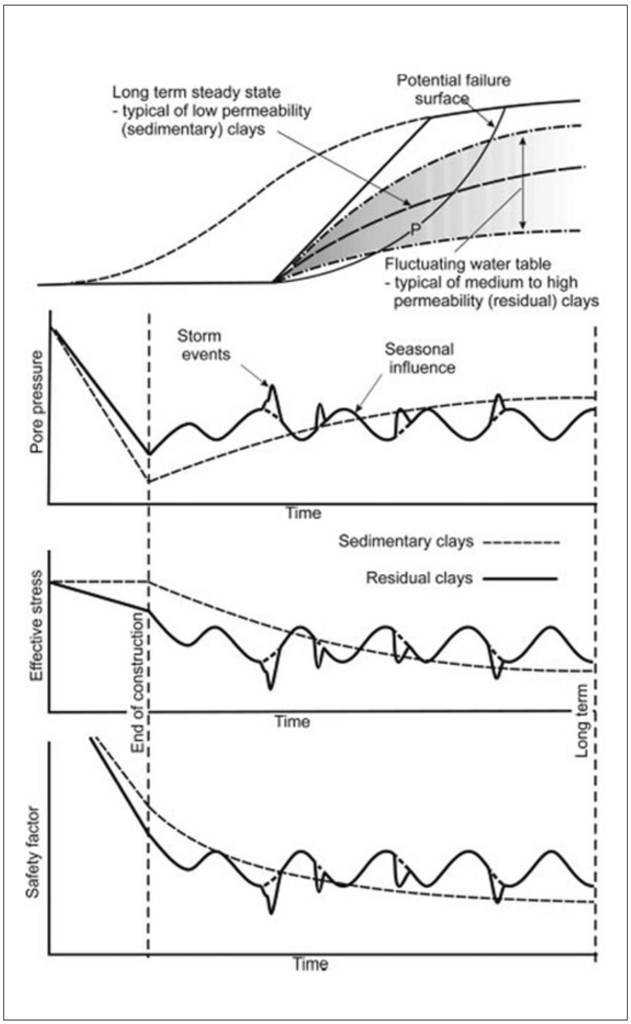


FIGURE 31: Conceptual illustration of the short and long-term stability of a cut slope in sedimentary and residual soil (from Wesley 2010).

Low lying areas consisting of fully saturated, low permeability sedimentary soils (e.g., Takaanini Formation southern lowlands) are more at risk of encountering failure in the long term (i.e., 5+ years post-construction), whereas cut slopes formed in residual soil are likely to encounter failure within a few years of formation of the slope. This is depicted in Figure 31, (extract from Wesley, 2010.)

An obvious strategy to mitigate pore pressure effects at the design stage would be to incorporate a high or worst-case groundwater table and drained parameters. Sensitivity to varying levels of saturation is also recommended and can be carried out using increasing Ru. Practitioners may refer to Unit 3 for a description of this method, and a wider discussion on appropriate methods for analysing high groundwater conditions..

Installation of subsurface drainage inhibits the rise of pore water pressures, thereby preserving effective stress, and is a valuable mitigation measure against longer-term cut slope failure. However, subsoil drains should be employed with caution in conjunction with other mitigation measures and are not recommended as a standalone solution in any but the lowest risk scenarios. These drains can become less effective over time due to blockage or damage associated with land movement, and where the geological model is not well understood and/or the drains are not suitably designed, they may not achieve the desired outcomes. For this reason it is recommended that the design analysis for transient groundwater conditions considers impaired drainage function, or complete drainage failure in the worst credible design case.

Suitable cut-slope angles vary significantly across the geological formations of Auckland, ranging from those that are unstable even at gentle gradients (e.g. Hukerenui Mudstone, Northland Allochthon) to those capable of maintaining stability at angles exceeding 30° (e.g. Awhitu Group).

Atkinson (2007) suggests adoption of critical state effective stress parameters (assuming c’=0 kPa) for the design of safe permanent cut slopes, which may be appropriate, particularly for high-risk slopes.

In the coarser grained units (e.g. Awhitu, Kariotahi Groups) longer slope lengths (corresponding to shallower gradient cut slopes) are more vulnerable to deep scour and erosion. This should be carefully considered when planning cut slopes in these soils, as although a shallower, more stable slope is desirable, it may develop erosion issues.

FIGURE 32: **Sub-vertical temporary cut slope in partly welded scoria overlain by ash and tuff in Auckland Volcanic Field prior to anchor installation. Performance of cut slopes in this geology is governed by blockiness, soil layers of different permeability, and groundwater levels.**

4.4.3 SAFETY BY DESIGN

Safety by design is discussed in detail in Chapter 9 of Unit 4, and includes guidance for implementing safety by design practices on small-scale projects which is useful for practitioners in Auckland where such projects are plentiful.

The ACCoP also includes guidance on safety by design (Section 2.4). For assets to be vested in Auckland Council ownership (and by extension, as recommended best practice for land development in the Auckland region), Auckland Council require that a risk assessment be undertaken for sites at risk from geohazards. The risks assessed should be communicated as a risk register in the reporting and on drawings submitted to Auckland Council for consent.

A simple risk assessment framework is provided in the Code of Practice for use, however in some cases alternative risk assessment frameworks may be more appropriate for the hazard being assessed (e.g., AGS 2007). Further discussion on risk analysis is presented in Part 6 of Unit 1. In cases where the untreated risk is Very High or Extremely High in accordance with the suggested risk assessment framework, Auckland Council are likely to require a Peer Review of the development proposal to confirm that the geohazard has been properly assessed and the mitigation measures are appropriate.

The risk register resulting from the earliest geohazard assessment, normally undertaken at due diligence or resource consent stage, forms the basis of the safety in design register and should be regularly reviewed and revised as the project develops through the design and construction phases.

4.5 MITIGATION MEASURES

Supported by administrative controls such as the ACCoP and the NZTA Bridge Manual, robust stabilisation measures are a requirement on most instability-prone sites in Auckland. Extensive information on the mitigation of slope instability is presented in Unit 1, Sections 10.4 to 10.7, and in Unit 4. Table 13 presents these mitigation measures in an Auckland context.

Suitable methods of mitigation of the slope instability hazard in Auckland encompass a broad range of measures, all of which may be effective, but depending on the development proposal, not all may be appropriate. Where space allows, conventional large-scale bulk earthworks operations to reshape the landform allow for remediation of and/or protection against land instability (Figure 33). Where the earthworks design is governed by geotechnical requirements including elements such as shear keys, retaining wall, buttress fills, and drainage designed to achieve acceptable factors of safety against instability, it is important that a specialist earthworks contractor with a proven track record in geotechnical stabilisation earthworks is engaged to complete the work. For urban properties where existing neighbours and structures present a space constraint, retaining wall approaches are preferred. A holistic design approach is of particular importance in coastal settings, where site-specific measures can sometimes be installed at the expense of the neighbouring properties for whom the effects of erosion are subsequently exacerbated.

Table 13: Typical Slope Instability Mitigation Measures

Effective & Widely Used Mitigation Measures	Advantages	Disadvantages	Widely Used in Auckland Terrane / Geomorphic Setting
Excavation and Earthworks Including: • Unloading • Shear Keys • Buttress Fills • Reinforced Earth/ Soil (RE/RS) Slopes • Regrading • Excavate and Replace	Cost effective Common practice in the Auckland region and generally well understood and implemented by experienced designers and contractors Can stabilise a large area of a site simplifying design Incorporates drainage, also lowering pore water pressures	Deep seated landslides may extend beneath shear key base Can trigger large landslides during excavation when poor construction practices are implemented (e.g. opening large widths of shear key prior to backfilling, excavation of toe of slope first before unloading the top) Unloading can have destabilising effect in both the short and long term Shear keys usually involve “keying” excavation into rock. Rock depths therefore are required to be relatively shallow. Additional consenting requirements may apply (e.g. Resource Consent) due to earthworks volumes, adding time and cost to projects. Construction may be limited to summer due to difficulty carrying out earthworks in wetter months	Widely adopted across all of Auckland. Waitematā Group, Takaanini Formation: Conventional bulk earthworks approaches are routine in this setting. Northland Allochthon: Is successfully implemented but requires careful design and construction consideration for both temporary and long-term stability outcomes.
Surface Drainage • Dish drains / Swales • Culverts	Cost effective Low complexity, can be installed by most contractors	May require specialist design to accommodate peak surface water flows May not be effective in isolation	Most beneficial to granular soil units such as West Coast Dune Sands and Auckland Volcanics where infiltration of surface water or surficial erosion is a primary landslide initiating factor.
Subsurface Drainage • Counterfort drains • Relief or dewatering wells • Horizontally drilled drains	Can be cost effective Can be applied to all soils as a means to increase resistance via increasing effective stress. Effectiveness may increase in residual soils where permeability is controlled by defects. Intercepts target layers (e.g., weak layers, shear planes, zones of high porewater pressure). Cycles of wetting and drying can reduce strength of sedimentary rocks. Drainage can keep groundwater conditions consistent minimising effect of rising / lowering groundwater tables (e.g., in sea cliffs).	Target strata can be critical and if not reached or sufficiently penetrated, the value of the drain is reduced. Effect of drainage can be low or overestimated in very low permeability soils. Can become blocked or deteriorate over time, compromising their function. Beneficial effect of drainage is often ignored due to gaps in knowledge of hydrological conditions. Investment in drainage may therefore not contribute to meeting design criteria. May require regular ongoing maintenance post-construction to be effective	Applicable to all geological settings and should be considered in conjunction with other mitigation measures as a combined approach. Drains should seek to target: • Weak zones, and layers of higher permeability confined by an underlying low permeability layer, e.g. completely weathered silt/ sand layers at the soil/ rock interface (refer to case studies in Section 5). • Failure surfaces and tension cracks in existing landslides. • Seepages and springs.
Reinforcing Piles • Palisade walls • Rigid inclusions	Reduced risk of triggering instability during construction vs. alternative methods (e.g. shear key excavations) Can be constructed during winter May have fewer consenting requirements vs. earthworks alternatives. Can stabilise deep landslides	Can be expensive due to the large shear and bending demands Generally need to be socketed into rock meaning pile lengths can be significant Can require multiple rows of piles to achieve factor of safety requirements Holes often collapse requiring installation of temporary or permanent casing, increasing costs.	All geologies / settings

Effective & Widely Used Mitigation Measures	Advantages	Disadvantages	Use in Auckland Geology
Retaining Structures • Anchored slopes • Anchored walls • Gravity walls • MSE walls • Embedded walls (soldier pile wall, cantilever wall)	Can maximise developable land area Can be constructed quickly Can incorporate a 'green' facing	Can be expensive Specific retaining walls (e.g. timber pole walls) may be more sensitive to slope movements or seismic shaking. Deep-seated movement may extend beneath retaining walls Often constructed to retain site boundaries requiring additional space (e.g. to extend anchors beyond boundary, temporary excavation behind wall) Can add complexity to projects Can increase consenting requirements Can appear 'hard', and not blend into landscape	All geologies / settings
Fences and Barriers Including: • Drapes • Attenuators • Rigid barriers • Flexible barriers	Cost effective Can be implemented in steep terrain where installation of other more robust methods are not viable. Can provide a mitigation to debris flow effects. Useful when space is limited (e.g. road corridors)	Does not prevent the rockfall / debris flow from occurring, only aims to stop debris. Hazard still exists. Ongoing maintenance requirement to check / remove debris. Barrier elements may need replacing. If excessive flow / fall volume, debris can overtop barrier Requires barriers to be positioned in the correct location	Employed in rockfall settings typically encountered in Waipapa Group, Waitematā Group, Waitākere Ranges, and Auckland Volcanics.
Vegetation / Erosion Control Matting	Cost effective Sustainable Multiple positive effects on stability (e.g. limiting of surface erosion, binds soil together increasing material strength, reduces pore water pressure generation, positive effects on creep, lowers debris flow risk)	May be ineffective to prevent instability alone Likely ineffective at preventing deep seated landslides May require maintenance May die in dry summers / floods Slope vulnerable prior to full establishment of vegetation	All geologies / settings
Avoidance or Retreat	Cost-effective Preserves natural environment Can be applied administratively, meaning physical works not required	Does not specifically improve the stability of the land Uncertainties in scale of instability may cause landslide to extend beyond zones of avoidance (e.g. excess runout, regression of head scarp) Renders parts of site undevelopable Not desirable for high density developments	All geologies / settings. Particularly effective in rural areas where land is zoned for low density development.

FIGURE 33: A conventional bulk earthworks site for residential subdivision in Auckland. A specialist earthworks contractor is recommended to manage a site of this nature, where installation of critical geotechnical stabilisation measures governs the civil earthworks design.

Careful consideration should be given to, among other things, the scale of a project, importance levels of structures, project budgets, proposed development densities, AUP zoning and objectives, critical design cases (e.g. temporary, permanent), and safety-in-design when selecting the most appropriate measure/s on a given project.

The mode of instability is also an important element to consider. Deep seated slope failures for example may be beyond the depth limit of many widely used mitigation measures, and deep piles or ground improvement methods may be the only appropriate options. Conversely, long term surficial soil creep may not require significant earthworks or installation of structural elements to rectify and planting with deep-rooted vegetation species (e.g. Vetiver grass) may be a suitable green alternative.

Some examples of mitigation measures which are used particularly frequently in Auckland include:

- Shear Keys – An effective mitigation measure for translational slides involving the excavation of material to intercept the governing shear surface(s) and replacement with engineered structural fill to buttress the slope. Ease of design, simplicity of construction, incorporation of drainage and lack of additional building consent requirements have contributed to shear keys being one of the most frequently used forms of slope stabilisation, particularly on earthworks projects. Excavation of shear keys comes with a high risk of initiating instability in the temporary case, often mitigated by limiting the length of open excavation at any given time and not carrying out shear key excavation works during winter.
- Palisade Walls – Some examples of shear key excavations in Auckland are shown in Figures 34 and 35. These are effectively a buried pile wall designed to intercept a shear surface at depth, generally with a design allowance for loss of support downslope of the wall due to future land instability. They are routinely employed where a bulk earthworks and shear key solution is not appropriate due to space constraints or the size of the project. These typically require a smaller workforce to construct when compared to bulk earthworks, can be installed during winter, and reduce the risk of initiating instability during construction. Palisade walls require additional structural design and consenting requirements and can be expensive. If groundwater or loose or soft soil layers are intercepted during drilling, hole collapse can make construction difficult and add time and cost to projects. As palisade walls can be installed at depth, they are also regularly used to protect coastal properties, as shown in Figure 36.

FIGURE 34: Shear key excavation in deeply weathered East Coast Bays Formation residual soil and very weak rock. Temporary stability of the excavation is managed by sequenced excavation and backfill placement over short sections. Fill wedge at left, intact ground yet to be excavated at right.

FIGURE 35: Hardfill being placed and compacted in a shear key excavation at the toe of a creeping translational block slide in Pakiri Formation.

FIGURE 36: Figure 36: Construction of a palisade wall with concrete capping beam in East Coast Bays Formation in a coastal cliff setting.

FIGURE 37: Tiered palisade walls to support building development on a steep slope in East Coast Bays Formation , with excavation after construction to support building development.

- Drainage – Elevated pore water pressures are a common cause of failure, and subsoil drains (counterfort, underfill, horizontally drilled) drains are therefore considered to be an important component of any effective mitigation measure. For a drainage network to be effective in low permeability soils, drain spacing should be reasonably close (see Fitch and Price, 2017 for guidance), and use of self-filtering drainage aggregates or fully wrapping drains in geotextile can improve drain performance over their design life. Drainage measures are most effective when they intercept the critical layer(s) on which sliding associated with elevated porewater pressures are predicted, and should also be extended to intercept tension cracks (where present) to capture infiltrating surface water. Safety is a key consideration, particularly when constructing counterfort drains, as the walls of deep vertical trench excavations can collapse prior to backfilling with drainage aggregates. The Health and Safety at Work Act 2015 and associated regulations, and Worksafe excavation guidelines should be followed at all times. Excavation sides should be benched (Figure 38) and personnel should never enter unsupported trenches.

FIGURE 38: Counterfort drain installation.

5 CASE STUDIES

The following case studies are presented to demonstrate real world examples of typical land instability in the Auckland Region, and practical approaches to investigation and analysis to assess their failure mechanisms and mitigate their effects.

5.1 KEPA ROAD LANDSLIDE

Orakei Basin is a well-preserved maar crater in the Auckland Volcanic Field, located on the eastern side of Auckland City. The suburbs of Remuera and Meadowbank are to the south, and Orakei to the north. Pourewa Stream passes through the northern side of the crater to Hobson Bay, which opens to the Waitematā Harbour to the west. The volcano erupted phreatomagmatically approximately 85,000 years ago, draping tuff and ash over the surrounding Waitematā Group landform.

The southern and western sides of the crater are characterised by a well-defined tuff ring forming sub-vertical bluffs up to 20 m high. The north-eastern side of the crater is known as the Pourewa Landslide Zone and is characterised by a geomorphically complex south-facing slope.

Four named landslides are mapped across this slope. From west to east, these are Ngapipi Road Landslide, Kepa Road Landslide, St Josephs Landslide, and Pourewa Landslide. The three western landslides are clearly identifiable by their large, well-defined head scarps. The Pourewa Landslide relief is more subdued, but no less significant (Figure 39). The landslide bodies demonstrate geomorphology typical of translational slides, comprising benches, sag ponds and hummocky ground (Figure 40).

FIGURE 39: Orakei Basin and the landslides of the north-eastern flank, labelled as follows: a) Orakei Basin, b) Ngapipi Road Landslide, c) Kepa Road Landslide, d) St Josephs Landslide, e) Pourewa Landslide. Base imaged sourced from Auckland Council GeoMaps, 1940 aerial photograph layer.

FIGURE 40: Kepa Road Landslide today.

These landslides are well known and well-studied, with the University of Auckland incorporating the site into field trips, geomorphological mapping training, and postgraduate research projects. The site lends itself to such study as it is largely undeveloped and presents as a classic example of planar sliding between geological units. Access to the study area is possible in consultation with, and with thanks to, Ngāti Whātua Ōrākei.

The Kepa Road Landslide is described as a translational block slide involving lithified tuff sourced from Orakei Basin sliding over the underlying East Coast Bays Formation (Waitematā Group) strata. The landslide debris is interpreted to be shallow, ranging from 1 to 3 m thick. It is agreed that a combination of a prevailing south-facing slope (favourable to sliding), high pore-water pressure at the contact between the draped tuff and the underlying residual clay soil, and removal of toe support over time by the Pourewa Creek at the toe of the slope led to the initial slide (Franklin, 1999; Brook, 2017). Secondary slope failures have occurred over time and in recent history, including two separate events on the southern side of Kepa Road in 1951 and 1984 (Brook, 2017). The landslide is still active, and recent UAV with Structure-from-Motion (SfM) photogrammetry studies have demonstrated creep within the landslide body (Brook, 2020). This is consistent with a history of tension cracks and deformation to Kepa and Ngapipi Roads. The ongoing creep movement is attributed to periodic elevations in pore-water pressure attributed to rainfall events (Liu, 2016).

The Kepa Road Landslide is a textbook example of translational sliding at the contact between two geological units. High pore-water pressures develop at the top of the residual clay layer in conjunction with rainfall events, initiating the primary failure event and triggering subsequent reactivation events and creep. This demonstrates the sensitivity of such slopes to rainfall, and highlights the potential for anthropogenic influences such as earthworks cuts and fills, and surface water discharge, to negatively impact the ongoing performance of natural slopes.

5.2 LANDSLIDE IN LAYERED ALLUVIUM

Fisher Parade is located on the eastern side of the Tamaki Estuary in the suburb of Sunnyhills. It is underlain by Takaanini Formation alluvium to a depth of approximately 25 m, with Waitematā Group below. In 1970, a large landslide occurred on the western side of Fisher Parade, impacting six properties who initiated claims for landslide damage compensation with the Earthquake and War Damages Commission (now NHC), who appointed Tonkin & Taylor Limited to undertake an investigation.

The preceding summer was notably one of the driest on record, leading to extensive soil shrinkage and fissuring across greater Auckland. Surface cracks allow for greater surface water infiltration than is normally possible, which, combined with high groundwater pressure conditions, can result in increased lateral forces contributing to landsliding and reduce the effective shear strength of the affected soils, particularly granular soils.

Tonkin & Taylor undertook site reconnaissance, and six boreholes were drilled for logging and laboratory testing. The original landform had an average inclination of 10°, which they noted is not normally a cause for concern. The landslide head scarp extended through the affected properties, and a block approximately 12 to 18 m wide moved down and west towards the shore, creating a shallow graben. Toe heave observed in the foreshore revealed steeply inclined peat beds and a pumiceous sand layer, leading to the postulation of a sliding wedge failure mechanism rather than a rotational slide.

The ground conditions were found to be reasonably consistent between test locations. A representative geological cross section through the landslide body (Cross Section D) is presented in Figure 41, extracted from the Tonkin and Taylor report (1970). The ground conditions are summarised using the language of the report, as follows (in stratigraphic order):

- Zone F: Miscellaneous fill attributed to the wider subdivision development, and landscaping activities.
- Zone E: Pale grey/brown, firm clay with average undrained shear strengths of greater than 8 psi (55 kPa).
- Zone D: Mixed layers of pumiceous silty sand, soft black organic silt, medium and firm clayey silts, with fine sand lenses in the organic silt layers.
- Zone C: Peats and organic silts with average undrained shear strengths of 6 to 10 psi (40 to 70 kPa) and occasional soft spots. A thin fine pumice sand band inclined towards the shore was logged. The Zone B contact below appears horizontal, while Zone D above is inclined.
- Zone B: Grey/green medium to firm silty clay with a thin organic silt bed.
- Zone A: Cemented Waitematā Group sandstone at approximately 25 m depth.

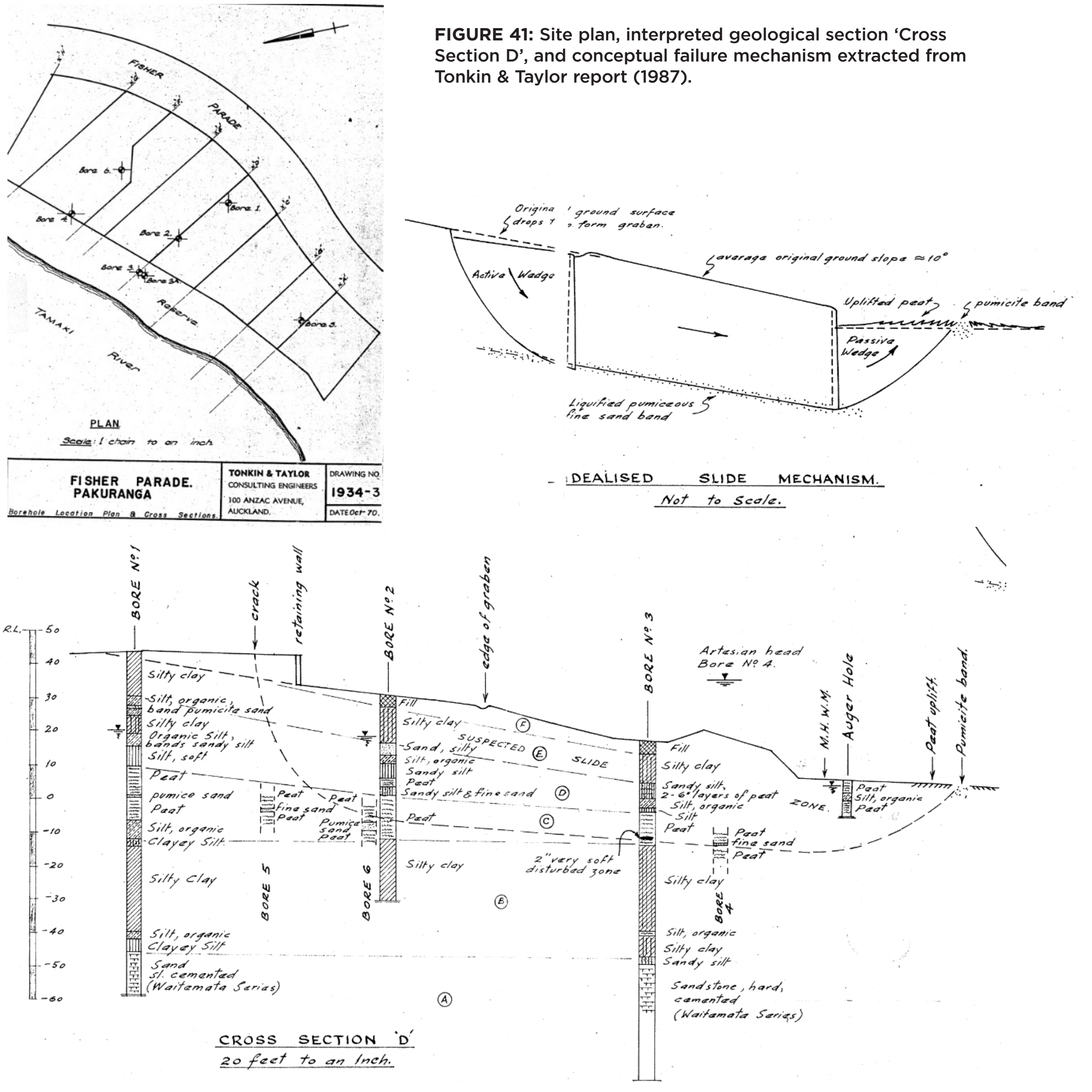


FIGURE 41: Site plan, interpreted geological section 'Cross Section D', and conceptual failure mechanism extracted from Tonkin & Taylor report (1987).

Significant drilling fluid was lost in Zones C to E, suggesting considerable fissuring in those layers. Piezometers installed indicated an artesian water head from Zone A. Laboratory testing, including ten consolidated undrained triaxial tests with pore pressure measurement, indicated that the alluvial soils were over-consolidated, showing evidence of effective cohesion.

The proposed failure model was established as follows:

- Pumiceous sand bands were observed in two of the boreholes in the suspected zone of movement (approximately 10 m depth), correlating to the foreshore observations, and in a third bore the area of interest was very soft and disturbed.
- Loss of drilling fluid in Zones C to E could be indicative of fissuring allowing water infiltration.
- Artesian pressure head in Zone A could have transferred up to Zones C to E if fissuring had also occurred in Zone B, which was considered possible due to the preceding dry summer.
- Increased porewater pressure in the pumiceous sand layers in Zone C could have induced a state of liquefaction (i.e., zero effective stress).
- Back analysis indicated that an average shear strength along possible sliding surfaces of 2 psi (13 kPa) was necessary to induce failure, which did not correlate with the numerous in situ strength tests in the boreholes. The triaxial tests were remarked on as having returned a “surprisingly high” average effective cohesion of 3 psi (20 kPa) for the peat layers.
- It was concluded that for a failure of the type modelled in Figure 41 to occur, an extensive surface with zero shear strength would be required. A net zero effective stress condition (i.e., effectively liquefaction) of the pumiceous sand layers in Zone C is mooted, influenced by the artesian head from Zone A and/or excess perched groundwater percolating through fissures in Zones D and E.

A conceptual mitigation measure was proposed, comprising installation of a series of pressure relief wells (15 no. x 75 mm diameter pipes installed at a 45° inclination to a depth of 30 m) to intercept the pumiceous sand layers in Zones C and D and relieve the excess porewater pressures, allowing the soil to re-establish shear resistance.

5.3 DEBRIS FLOW IN WEST AUCKLAND

In January and February 2023, Auckland experienced two extreme rainfall events following an unprecedented level of rainfall through the month of January. A total of 539 mm of rainfall was recorded at the Albert Park rain gauge for January 2023, which included the rainfall from the Auckland Anniversary Weekend storm (January 27) and was followed by landfall of ex-tropical cyclone Gabrielle (13 February). GNS have mapped approximately 50,000 landslides across the Auckland region (GNS, 2023). Recovery work is ongoing, and the geotechnical community is beginning to see increased publication of technical reports and knowledge sharing to inform our collective understanding of landslide hazards in Auckland.

This case study draws on the recently published work by Brook and Nicoll (2024), Roberts et al., (2024) and Howard and Roberts (2024), however work is ongoing. The purpose of this case study is to highlight the key observations and learnings published to date from the landslides that impacted the west coast communities of Auckland.

In Muriwai, part of the town sits downslope of an up to 80 m high escarpment up to 1.5 km long, formed in weakly cemented Awhitu Group sand and sandstone unconformably overlying Nihotupu Formation, forming a relatively impermeable contact (Brook and Nicoll, 2024). Rainfall associated with Cyclone Gabrielle initiated multiple large debris flows from the escarpment that inundated the community below, and tragically resulted in two fatalities. An emergency response involving Rapid Building Assessments (RBA) was undertaken for the affected area, relying on limited available data and adopting a Fahrböschung angle (F-angle) approach (Figure 42) based on seven mapped landslides within the escarpment area (Roberts et al., 2024). The adopted range was 22° to 25° taken from the escarpment crest (Howard and Roberts, 2024) and was used to rapidly characterise the wider area in the days following the event.

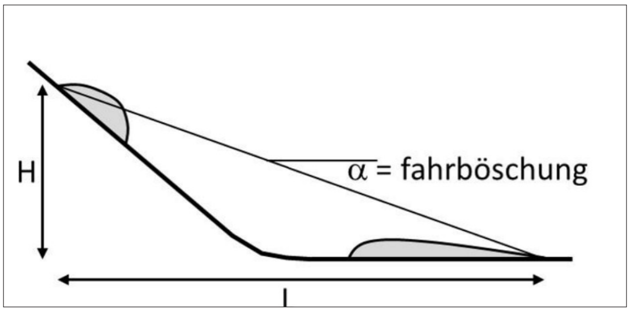


FIGURE 42: **Assessment of Fahrböschung angle (F-angle) taken from the crest of the source area to the toe of the runout extent, where H is the height of the slope and L is the length (Mitchell and McDougall, 2019).**

The landslides sourced from the escarpment in Muriwai were described as translational slides, or in some cases toppling of steep faces, that transitioned to debris flows as the landslide mass disaggregated (Howard and Roberts, 2024). Saturation of the shallow surface soil layer was the primary trigger, although the influence of springs at the toe of the slope and effects of shallow-rooted vegetation are also considered.

At Piha, the landslides were more discrete than occurred at Muriwai, but no less destructive (GHD, 2024). The steep, west-facing coastal slopes are up to 100 m high, and in many cases steeper than 35°. The source geology is of the Waitākere Group, primarily weathered Piha Formation, with outcrops of Waitarua Formation at the northern end of Piha (Figures 43 and 44). The land at the toe is underlain by Holocene-aged sand dunes that are locally overlain by and interfingered with colluvium. Work by GHD in the Piha community identified a translational slide failure mechanism in natural slopes that in some cases transitioned to a debris flow. The landslides were typically shallow, involving the 0.5 to 1.5 m thick soil layer overlying less weathered strata. A majority of the recorded flows were between 50 and 100 m^3. In contrast to the landslides at Muriwai, the Piha debris flows were more channelised. The interpreted trigger mechanism was saturation of the shallow surface soils and concentrated surface water flows.

Of note in Piha was a secondary rockfall hazard that was identified after the initial landslides, due to new source areas formed by head scarps and debris hung up on slopes. Site reconnaissance by a Unit 7B.1 author into the slopes above north Piha identified older rockfall source areas and associated blocks up to 1 m diameter on the slope, obscured by the dense vegetation. These were interpreted to predate the 2023 event. In some cases the blocks had reached the developed areas, and in some cases had been used in landscaping by the property owners.

After the emergency response, GHD were commissioned by Auckland Council to undertake a risk assessment for landslides originating from the Muriwai escarpment (Howard and Roberts, 2024). Their work is presented in Issue 108 of NZGS Geomechanics News (December 2024), and the notable findings are as follows:

(FIGURE 43 AT LEFT) Translational slide that transitioned to rock avalanche and debris flow, head scarp and flow path (post-demolition). Head scarp at ridge crest exposes a faulted contact between Piha Formation and Waiatarua Formation.
(FIGURE 44 AT RIGHT) Debris fan and impacted dwelling (pre-demolition), debris comprises tabular Waiatarua Formation gravel and cobble sized fragments sourced from the highly fractured rock mass at the head scarp location.

- **Desktop study of LiDAR survey data, historical aerial photographs and landslide records:** In 1965 similar landslides resulting in two fatalities impacted two residential properties below the escarpment. Review of historical aerial imagery was of limited use in identifying large historical landslides, including the known landslides of 1965. This was attributed to rapid regeneration of vegetation across the slopes after landslide events.
- **Engineering geological mapping of the escarpment and recent landslides:** The topography of the escarpment and land below is not particularly incised by surface water flows, meaning the debris flows were not well-channelised. Flow directions were primarily influenced by slope orientation. Large trees were entrained in many flows, attributed to increased destructive power. A detailed assessment of 32 landslides identified a large range in F-angles of 16° to 42°, in contrast with the angle of 22° adopted for the RBA process. This variation is attributed to variability in topography and surface obstructions (e.g., vegetation, buildings). Closer agreement in F-angle was found when compared to the Hunter and Fell (2002) method.
- **Site investigation involving rotary cored machine boreholes with in situ and laboratory testing, and installation of piezometers:** The escarpment stratigraphy comprises a layer of massive to thickly bedded, extremely weak to weak sandstone, with occasional layers and lenses of silt, clay and peat. Testing over the upper tens of metres of the escarpment returned SPT N-values <20, and UCS <1 MPa. Deeper layers were more cemented, returning N >50 and UCS 1-2 MPa. Colluvium was encountered at the toe of the escarpment, including entrained sandstone blocks, attributed to a long history of landslides.

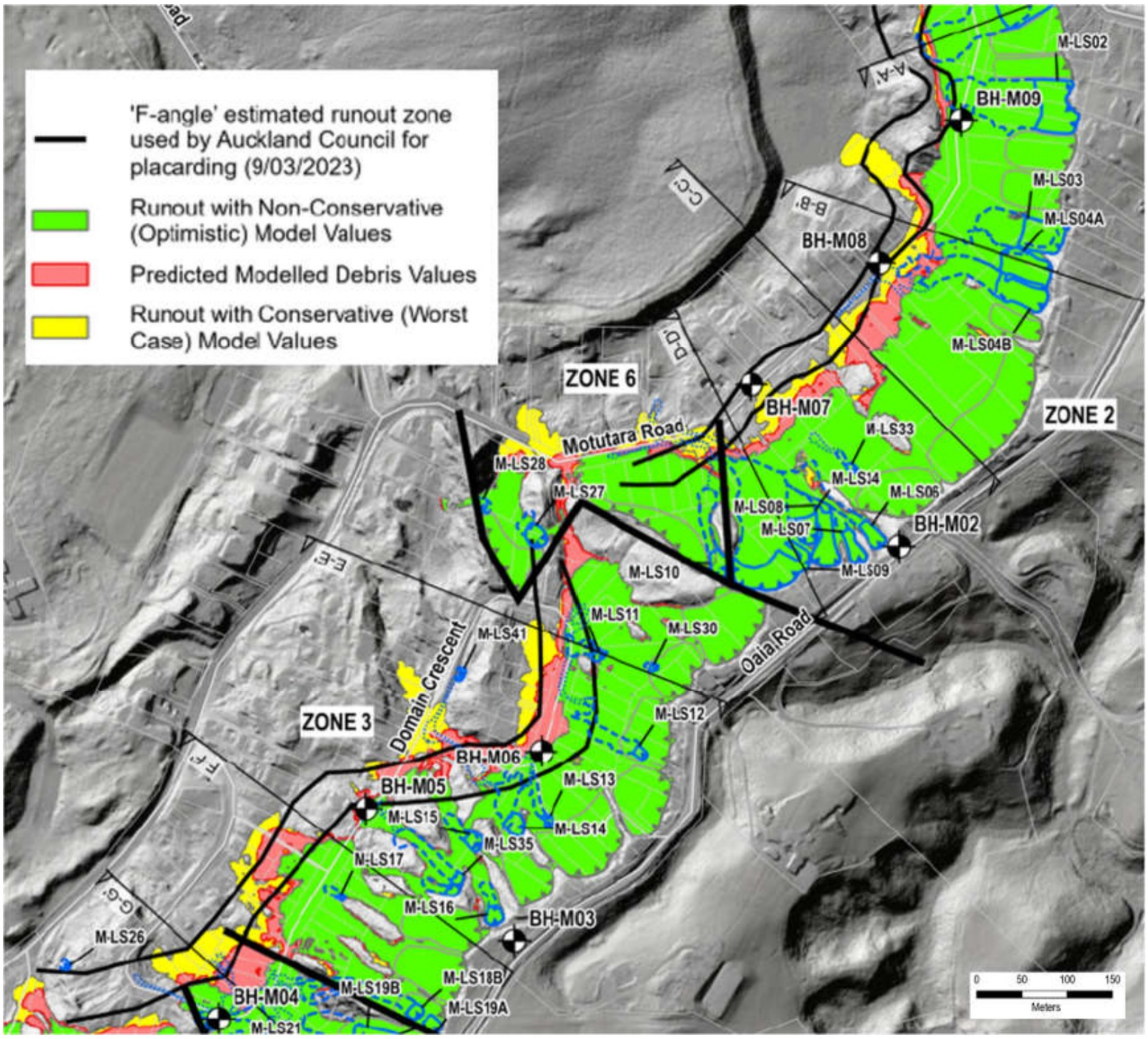


FIGURE 45: Comparison between predicted best- and worst-case F-angles used in Rapid Building Assessment and RAMMS model by GHD (from Roberts et al., 2024)

- **Two-dimensional limit equilibrium slope stability analysis and three-dimensional debris flow analysis using RAMMS software:** A simplified back-analysis of the escarpment identified an effective cohesion (c') of 21 kPa and angle of internal friction (ϕ') of 39° for the Awhitu Group. Calibration of the three-dimensional RAMMS model with the observed landslide characteristics at each analysis location was critical to allow the model to account for variation in debris and terrain effects when used to predict future events in similar settings. Simulated small debris flows travelled further than that observed, which is attributed to the effects of surface obstruction and variability in source material volume. The debris flow extent predicted by the RAMMS model was found to be broadly consistent with that adopted by Auckland Council for the RBA process (Figure 45), confirming the suitability of empirical F-angle approaches to rapid risk assessment in emergency response events.
- **Qualitative Risk Assessment:** As well as that for Muriwai described in the paper referenced for this study, GHD have undertaken quantitative risk assessments on behalf of Auckland Council for other west coast communities using the AGS (2007c) framework. Their work has informed practitioners undertaking such assessments on behalf of Auckland Council across the region, most notably with regard to the V($_{D:T}$) parameter considering occupant vulnerability.

Many houses in Auckland are conventional lightweight timber framed buildings, and a clear difference in structural performance is observed between inundation and evacuation scenarios. In the case of Muriwai, inundation of such a building by as little as 0.5 m height of debris was sufficient to cause partial or total collapse in some cases. Survivability of an occupied building in this scenario is considered to be low. There is clear variability between cases and the effects of vegetation entrained in debris are acknowledged as worsening the impact. However, it is important to note that of all of the landslides that occurred in the Auckland Region during the MORLE of 2023, the three fatalities occurred in cases of landslide inundation (two in Muriwai, and one in Remuera in East Coast Bays Formation). No fatalities occurred in scenarios where the landslide was downslope of the affected building.

5.4 CUT BATTERS IN WAITEMATĀ GROUP

East (1974) presented an inclined plane slope model for cut slopes in Waitematā Group sediments, where the residual soil overlying the rock head grades from predominantly sandy soils near the surface to predominantly silty sand soils at the soil rock contact (allowing for local variations and the heterogeneity that arises from a vertical weathering profile in an inclined bedded rock).

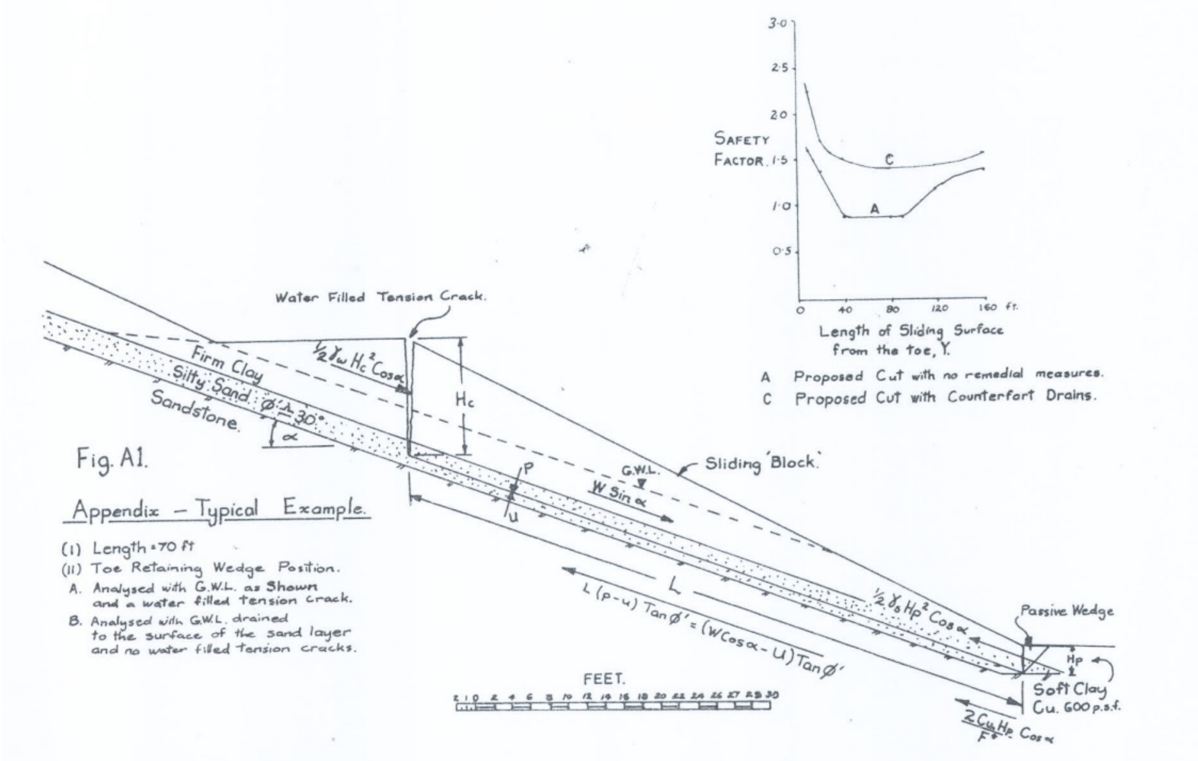


FIGURE 46: A conceptual drawing of the sliding block on an inclined plane (wedge method) model adopted by East for failures of cut slopes in residually weathered Waitematā Group soils.

In a typical construction case considering excavation to form a cut batter, the work results in the following:

- A reduction of normal stress (p) on the remaining soils, resulting in a reduction of shear strength. $S = (p-u) \tan \varphi'$, $(c'=0)$
- Removal of some (if not all) of the toe of the slope, removing the passive support to resist sliding

East presented three case studies from cut slopes in Auckland in which excavation resulted in failure, with discussion on the analysis and remediation measures undertaken and their variable levels of success. The practical observations and conclusions are summarised below. Although computational analysis approaches have progressed significantly from the hand calculations adopted for this case study, the findings are no less relevant today and demonstrate the value of a simple approach for simple slopes.

The three case studies include cut slopes to form two school fields (Massey High School and Selwyn College) and a cut platform for a power substation in Henderson. In all cases a cut slope was formed in residually weathered Waitematā Group soils, and in all cases a slope failure followed within 1 to 5 years. The following mitigation measures were employed:

- Massey High School – Counterfort Drains – Successful
- Henderson Substation – Bored Horizontal Drains – Successful
- Selwyn College – Rock Toe Without Drainage – Unsuccessful

Observations from the three case studies present the following consistencies:

- The timing of the major failure occurs in the winter months, and generally 18 months following construction (although it was 5 years in the Henderson Substation case).
- The failure is slow, occurring over one to two days, with subsequent creep initiated by rainfall events.
- Tension cracks and grabens develop at the crest of the slope, and toe rupture is observed where the rock is daylighted in the lower portion of the excavation.
- Water may be observed in tension cracks and at the toe of the excavation, and piping of silt and sand soils may occur.

Site investigations are undertaken for the slips, comprising the following:

- Machine and hand-held drilling equipment is used as appropriate for the setting, and DCP testing in situ.
- Simple piezometers are installed, with monitoring data confirming that typical winter groundwater levels are within 1 m of the ground surface and sometimes artesian in the lower reaches of the slope. In summer a 2 to 3 m drop in water levels is observed.
- Innovative low-cost ‘slip plane indicators’ are employed to assess the depth of sliding, with a correlation to sliding at or immediately above the soil/rock contact within the silt and sand layers.
- Typical properties of the clayey silt and silty sand soils are obtained through conventional testing measures:

$C' = 0$	$\phi' = 28° – 32°$	Cone Resistance = 20 – 40 kg/cm^2
Cu = 25 – 35 kPa	$\gamma = 17$ kN/m^3	W% = 30 – 40%
PI = 20%	LL = 45%	PL = 25%
<2μ = 20% (clay fraction)	<60μ = 80% (silt fraction)	K lab = 10^{-4} mm/sec The relatively low permeability is attributed to the high groundwater levels.

The role of the high groundwater level within the slope, and its effect on the near-cohesionless silty sand and sandy silt layer at the inclined soil/rock contact, is identified as the primary driver towards the observed slope instability. In the period following excavation, when some of the normal stress on the cohesionless layer is removed together with some of the toe support, creep within that layer may initiate and tension cracks develop. In winter months, high groundwater levels increase porewater pressure within the cohesionless layer resulting in a reduced shear strength and further creep. This may continue seasonally, gradually worsening over time. Failure initiates when the combined effects of high porewater pressures within the cohesionless layer and the driving force of a water filled tension crack are realised.

In the case of inclined plane failures discussed by East, drainage measures to reduce the porewater pressure within the cohesionless layer have the most significant impact on factors of safety. It is considered essential that the drains intercept the tension cracks to mitigate surface water infiltration leading to the build-up of forces on the sliding block (Figure 46).

Acknowledging the simplified slope model used in these case studies, East notes that reduction of water pressures in the cohesionless soils and in the tension cracks will improve the factor of safety for the slope even where the properties of the failure plane are incorrect.

In the Selwyn College case study, the failure is concluded to be a reactivation of a previous landslide. Although the analysis demonstrated that drainage was a necessary measure to achieve acceptable factors of safety and recommendations to that effect were given, the constructed remedial measure comprised a ‘rubble rock’ shear key at the toe and no drainage. The shear key did not intercept the slide, rather being installed in front of the toe, and as such the landslide reactivated almost immediately.

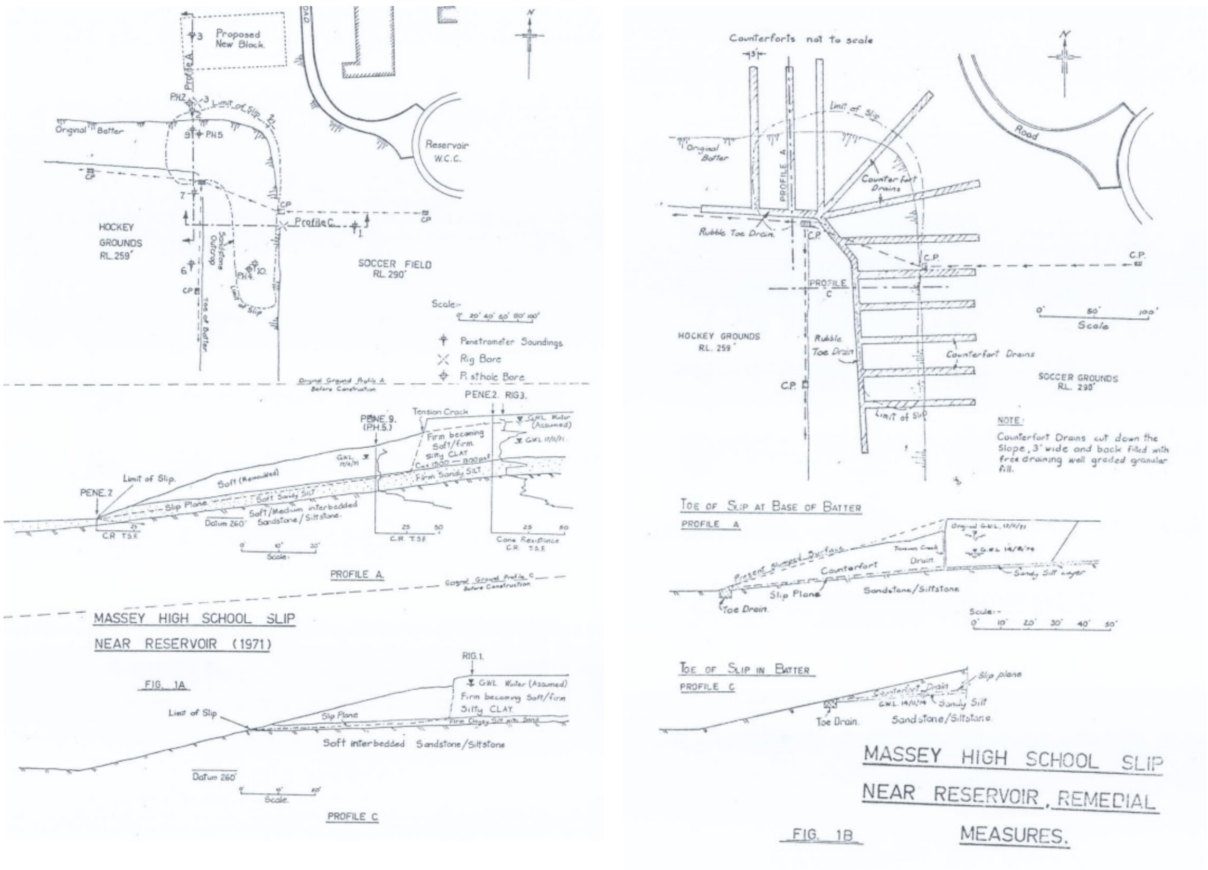


Figure 47: Massey High School failure conditions and remediation measures. Note in Figure 1B the counterfort drains extend beyond the mapped tension cracks.

The following conclusions are presented for practitioners:

- Where earthworks are undertaken in residual soils similar to that of the Waitematā Group, an investigation should be designed to determine the depth to and inclination of the soil/rock contact, and the nature of the soils immediately above that contact.
- If cut batters are to be formed in such soils, and an inclined plane with a block slide failure model is considered, drainage to suppress and control local groundwater levels is likely to be an appropriate and cost-effective mitigation measure.
- Slides on an inclined plane at or less than 10° is possible.
- Drainage such as counterfort drains or horizontal bored drains are proven effective measures provided they intercept the layer of suspected failure and all known and suspected tension cracks. Continuous pipes should be used.
- The analysis and remediation approaches are considered appropriate for any layered soil setting with a hard layer interface. In primarily cohesive soils, the groundwater response to drainage will take longer than cohesionless soils.
- Stability analysis should consider the residual angle of friction for the failure layer (φ'_r), particularly if previous movement is suspected.

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