REPORT

The Point Mission Bay

Geotechnical Assessment Report

Prepared for

Ngāti Whātua Ōrākei Whai Rawa Ltd and Generus Living Group

Prepared by

Tonkin & Taylor Ltd

Date

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1 Introduction

Tonkin & Taylor Ltd (T+T) has been engaged by Generus Living Group Ltd (Generus) and Ngāti Whātua Ōrākei Whai Rawa Ltd to undertake geotechnical investigations and assessment for the proposed development of the retirement village referred to as 'The Point Mission Bay' (TPMB).

The purpose of this report is to provide a summary of all investigation work completed and an overview of the inferred geotechnical model of the site. This report also outlines the geotechnical assessment and recommendations for the proposed development.

This work has been provided in accordance with our offer dated 12 March 2025[1]. The scope and objectives of this assessment are summarised below:

1. Review previously undertaken geotechnical investigations at the site.
2. Undertake geotechnical investigations, including 3 No. rotary cored boreholes and 4 No. hand augered boreholes.
3. Installation of groundwater monitoring standpipes and monitoring of groundwater levels using automated logging devices.
4. Develop a ground and groundwater model for the site, including preparation of a geological section to present the inferred ground and groundwater profile in relation to the proposed basement levels.
5. Assessment of the following geotechnical issues with respect to the proposed development:
   - Review the basement retention scheme to assess the stability of temporary batters to support the basement excavation, or the requirement for temporary/permanent retention.
   - Retaining wall analyses to provide preliminary design of any temporary/permanent retention and assess potential ground deformation due to retaining wall movement.
   - Groundwater drawdown assessment in relation to Auckland Unitary Plan (AUP) considerations.
   - Review ground settlement from combined actions of dewatering and retaining wall deformation (if applicable) and assessment of effects on neighbouring buildings and services.
   - Assess preliminary foundation options to support the structures, including shallow raft foundation spring stiffnesses for preliminary design and piled foundations.
   - Assess slope stability of the site in accordance with Chapter 36 of the AUP Guideline.
6. Preparation of a Geotechnical Report suitable for a substantive Resource Consent application under the Fast-track Approvals Act 2024.
7. Preparation of Draft Groundwater and Settlement Monitoring & Contingency Plan (M&CP).

[1] T+T (12 Match 2025), Letter to Generus Living Group Ltd The Point, Mission Bay – Variation 01, Geotechnical Assessment for Resource Consent. Ref: 1098049.

2 Background

2.1 Site description

The site is located in Ōrākei, Auckland, approximately 600 m south-west of Mission Bay Beach.

An aerial image of the current site layout is presented below on Figure 2.1.

The site is approximately 24,341 m^2 and is currently a mixture of vacant land and residential land use, consisting of two Eastcliffe Retirement Village apartment complexes in the eastern portion of the Site (proposed to be demolished), and the aged care facility in the west (to be retained and integrated with The Point. The vacant (central) part of the Site previously consisted of retirement apartment blocks, which have since been demolished.

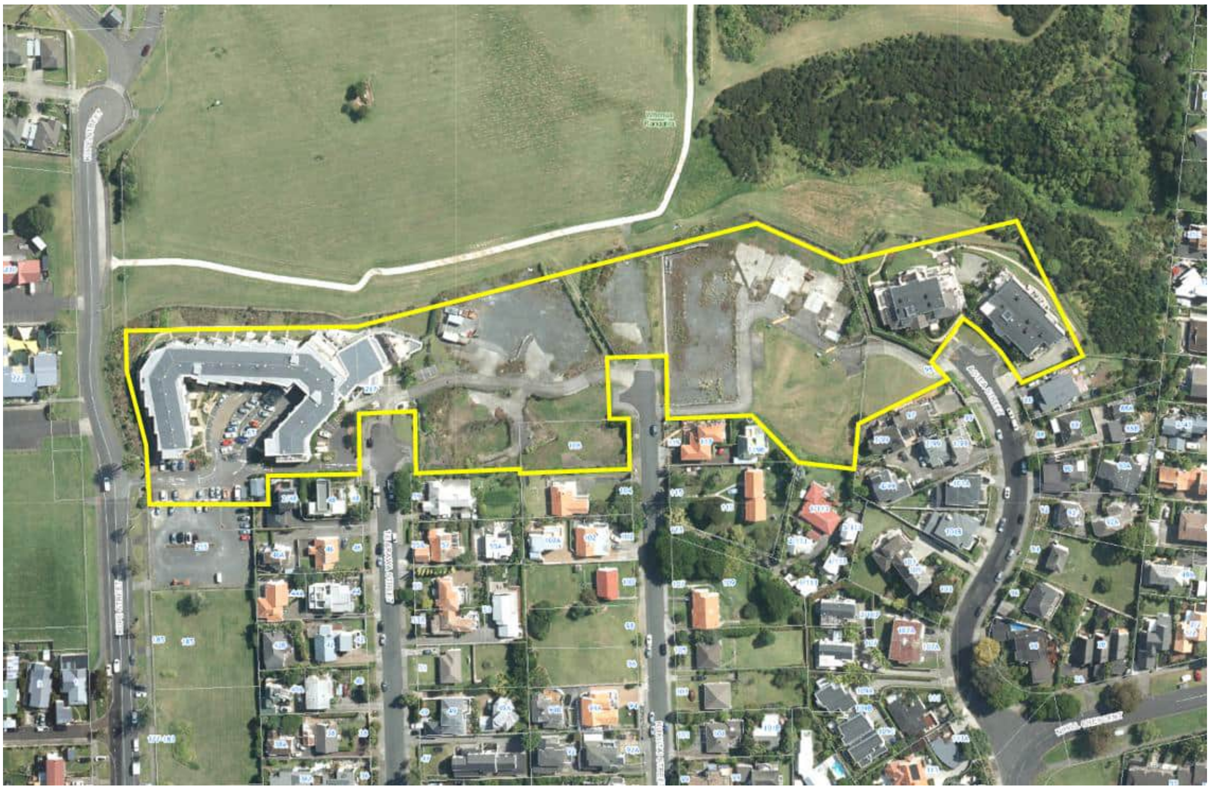


Figure 2.1: Site layout plan, approximate site boundary shown in yellow

The site is located in a residential area and is surrounded typically by dwellings of 1 to 2 storeys, excluding the three-storey retirement village building at the western end of the site, which will be retained and refurbished, and Takaparawhau / Michael Joseph Savage Memorial Park to the north.

The site is generally flat to gently sloping at an angle of less than 5-10 degrees falling from west to east. A series of 30-40° slopes with benches are present near the centre of the site at the eastern edge of the former terraced housing (recently demolished). These are man-made cut / fill benches associated with the former development.

Beyond the site to the east, the ground profile falls at approximately 15-20° towards the inland coastal cliffs, approximately 40 m north-east of the site boundary.

2.2 Proposed development

As shown in Figure 2.2, the proposed development involves the construction of 5 buildings across the site ranging in height between 5 to 7 stories, which includes partial basement levels of 1 to 2 stories that daylight from the sloping ground across the site that falls from west to east. A shared basement is proposed between Buildings 2, 3 & 4. The area between the buildings is to be used for recreation and hard and soft landscaping.

In accordance with the preliminary development plans provided[2], the lowest proposed floor levels across the building basement are summarised below:

- Building 1: RL 16.5 m
- Building 2: RL 19.95m to 28.3 m
- Building 3: RL 28.3 m to RL 31.75 m
- Building 4: RL 31.75 to 35.2 m
- Building 5: RL 35.2 to 39.2 m

Our assessment has allowed for bulk excavation up to 1.0 m below the finished floor levels to account for the floor slab and subsoil drainage construction. On this basis, temporary retained heights of up to approximately 8.75 m are required to support the proposed excavation.

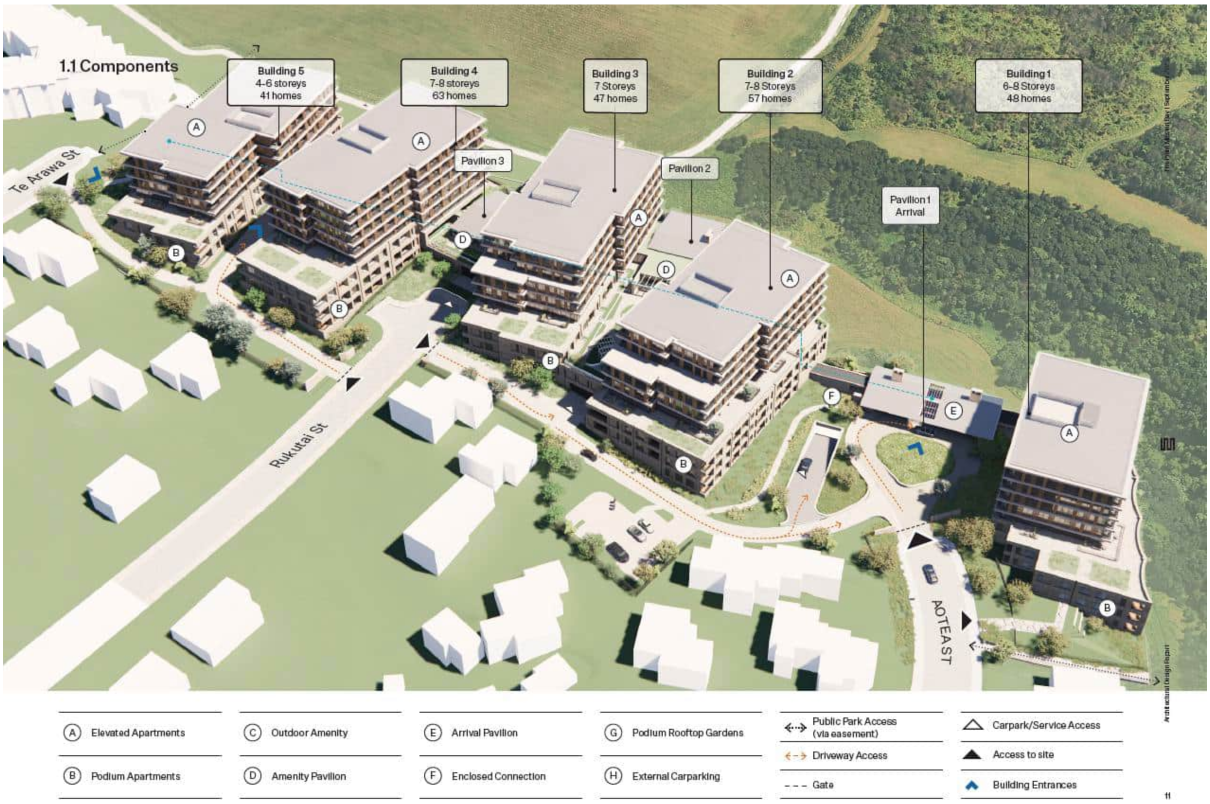


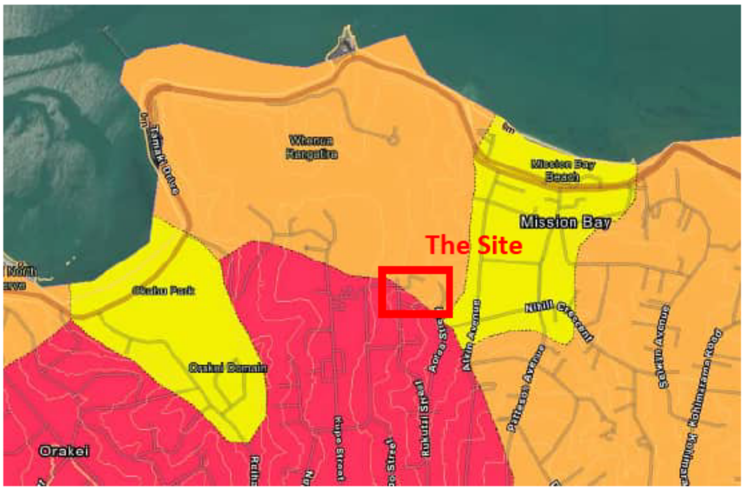
Figure 2.2: Site masterplan of proposed development (looking north-west)

[2] Warren & Mahoney (8 November 2025). Architectural Design Drawings Prepared for Ngāti Whātua Ōrākei Whai Rawa Ltd and Generus Living Group, Rev B. Job No. 10476.

2.3 Published geology

Published geological information[3] (Figure 2.3) indicates the site is located on the inferred boundary between sedimentary weak sandstone of the East Coast Bays Formation, comprising alternating sandstone and mudstone with variable volcanic content, under the north-eastern portion of the site, and igneous extrusive sediments comprised of Lithic Tuff of the Kerikeri Volcanic Group under the south-western portion of the site.

ECBF rock typically comprises very weak muddy sandstone and mudstone, typically exposed within the sea cliffs along Auckland's north shore and eastern suburbs. This rock mass typically weathers insitu to form stiff to hard silts and clays. This unit is anticipated to underlie the majority of the site and is also expected to underlie any volcanic deposits present.



Legend	
avt	Lithic Tuff
ta	Undifferentiated Alluvium
re	East Coast Bays Formation

Figure 2.3: Published geological map

2.4 Previous geotechnical investigations

Aurecon has previously undertaken geotechnical investigations at the site in 2018 and 2019, which are presented in their 'Phase 1 GIR' dated June 2019[4]. These investigations were undertaken for a previous development scheme and were limited to the central parts of the current development site within the approximate footprint of Buildings 2 to 4, as shown on the site plan in Appendix A. The investigations are summarised below, with investigation logs presented in Appendix B.

<u>2018 Aurecon Investigations</u>

The investigation comprised of the following:

- 6 No. machine drilled boreholes (BHs).
- 12 No. Cone Penetrometer Tests (CPTs).
- 4 No. groundwater monitoring standpipes.

The investigation logs provided in the 2019 Aurecon report are limited to those located within the 'Phase 1' area of their development, which includes only two boreholes (BH05 & BH06) and seven CPTs (CPT06 to CPT12). The locations of the remaining boreholes and CPT tests have not been provided and their location is unknown.

[3] Kermode, L.O. (1992). *Geology of the Auckland Area.* Institute of Geological and Nuclear Sciences Ltd.

[4] Aurecon (10 June 2019). Report to Ngati Whatua Orakei Whai Rawa Limited. *Eastcliffe Retirement Village – Phase 1. Geotechnical Interpretative Report.* Ref: 255232.

<u>2019 Aurecon Investigations</u>

These investigations were undertaken between February and March 2019, comprising the following:

- 10 No. machine drilled rotary cored boreholes to depths of between 11.1 m and 24.6 m below ground level (bgl).
- 6 No. standpipe installations for groundwater measurements, with a typical screened depth of between 3 m and 6 m bgl.
- Groundwater monitoring of the above standpipes monitored manually at monthly intervals between February 2019 and April 2019.

2.5 Project-specific geotechnical investigations

T+T completed site-specific geotechnical investigations for the proposed development between 9 June and 11 June 2025. This consisted of the following:

- 4 No. hand auger boreholes up to approximately 3.8 m depth;
- 2 No. machine boreholes ranging in depths from 9.1 m to 13.8 m;
- 1 No. wash drilled machine boreholes to approximately 7.5 m depth; and
- 3 No. dual-nested standpipe piezometers installed within BH01, BH01A and BH02.

The boreholes were drilled by DCN Drilling Ltd under the supervision of a geotechnical engineer from T+T. Locations of the machine boreholes are presented in Appendix A with borehole logs and photographs of the recovered core presented in Appendix B.

Table 2.1: Summary of T+T site-specific geotechnical investigations

BH ID	Location (NZTM)(1)		Ground Surface elevation RL (m) (1)	Termination depth (m bgl)	Reason for termination
	Northing	Easting			
BH01	5920109	1762714	38.3	9.1	Target Depth
BH01A	5920110	1762713	38.3	7.5	Target Depth
BH02	5920110	1762965	16.6	13.8	Target Depth
HA01	5920058	1762661	45	3.0	Target Depth
HA02	5920113	1762605	45	3.0	Target Depth
HA03	5920070	1762723	34	3.0	Refusal
HA04	5920141	1762939	16.7	3.8	Refusal

(1) Recorded using a handheld GPS unit with a reported accuracy of ± 2 cm.

Table 2.2: Summary of T+T site-specific geotechnical investigations

Piezometer ID		Collar RL (m)	Piezometer Screen Installation RL (m)	Type	Geological unit over screened depth
BH01	Shallow	38.3	33.3 to 35.3	Standpipe	Residual ECBF
	Deep		29.3 to 31.3		SW to UW ECBF Rock
BH01A	Shallow	38.3	35.3 to 37.3	Standpipe	Residual ECBF
	Deep		31.3 to 33.3		SW to UW ECBF Rock
BH02	Shallow	16.6	11.6 to 13.6	Standpipe	Residual ECBF
	Deep		3.1 to 5.1		SW to UW ECBF Rock

2.6 Subsurface conditions

The subsurface model presented in this report has been inferred using the historical and site-specific geotechnical investigations outlined above. The nature and continuity of the subsurface conditions away from the investigation locations are inferred; however, it should be appreciated that actual conditions can vary from those presented.

The subsurface conditions encountered at the site typically comprise:

- A surficial fill layer of variable composition, including well-graded gravel typically encountered at the surface and underlain by reworked cohesive fill. The thickness of this fill varies across the site, with the greatest thickness encountered at the man-made benches formed across the site and the existing Aotea Apartments.
- This overlies a sequence of weathered East Coast Bays Formation (ECBF) soils and rock, which generally comprises stiff to hard clays grading to very weak to weak rock at depths of between 6.0 and 13.5 m below ground level.

A summary of the geological units encountered is presented in Table 2.3 below. A geological cross-section through the site from west to east, and an ECBF rock surface isopach plan are included in Appendix A.

Table 2.3: Summary of ground conditions

Geological Unit		Typical Description	Top of the geological unit		Thickness range (m)	Strength (1)
			Depth (m bgl)	RL (m)		
Fill		Variable fill, comprising silt and gravelly silt; brown / grey. Stiff to hard.	0.0	14.6 to 40.1	0.1 to 2.8 (3.8) (2)	N = 2 to 3 SV = 45 to UTP q_c = 1 MPa
East Coast Bays Formation	Residual soil	Stiff to hard SILT / CLAY mixtures; light grey/orange brown becoming grey.	0.1 to 2.8(2)	23.4 to 39.1	1.0 to 6.4	N = 5 to 19 SV = 62 to UTP q_c = 1 to 2 MPa
	Weathered ECBF	Highly to moderately weathered, grey, Siltstone / Sandstone mixtures, consisting of medium dense/hard soil deposits.	1.6 to 7.8	10.5 to 30.5	2.0 to 8.1	N = 19 to >50 SV = UTP q_c = 4 to 8 MPa
	ECBF rock	Slightly weathered to unweathered, grey, Sandstone / Siltstone. Very weak to weak	3.0 to 10.5	6.2 to 34.2	Not confirmed	N > 50 q_c = refusal

1 SV = Shear strength recorded by hand vane (corrected for spring constant)
SPT = Standard Penetration Test (number of blows to penetrate 300 mm)
q_c = CPT cone resistance (typical range within the layer)

2 Single borehole location (BH48) with significantly greater thickness of fill outside of typical range.

2.7 Groundwater

The Auckland region is broadly characterised by transient (perched) groundwater levels within near-surface deposits and a deeper groundwater level within the ECBF. Groundwater behaviour is significantly affected by a combination of the local topography, the underlying geological units, and the surface-built environment.

The groundwater at the site is influenced by the inland coastal slopes approximately 40 m north-east of the eastern boundary, with a toe elevation of approximately RL 5 m. Coastal slopes within the Auckland region are generally observed to have a regional groundwater table at or near the toe of the slope profile, with groundwater rising inland typically observed at gradients in the order of 2-5%. Based on this relationship, the regional groundwater level at the eastern edge of the site is between approximately RL 6 to 7 m, rising towards the west.

The groundwater profile around the development comprises a perched groundwater system, overlying a deeper, more continuous regional groundwater level. The perched groundwater is predominantly occurring within the residual ECBF and weathered ECBF, constrained by local underlying low-permeability layers, and occurs as discontinuous lenses or 'pockets' rather than extensive connected layers.

This has been confirmed by the dual-screen standpipe piezometers that were installed within the 3 No. machine boreholes following completion of the investigation to evaluate the perched and regional groundwater levels at the site. The shallow screens are located within residual and weathered soil layers to capture the 'perched' groundwater profile with depth, with the lower screens located within the underlying slightly weathered to unweathered ECBF rock to measure the 'regional' groundwater table.

Monitoring of these piezometers was undertaken between 13 June and 27 June 2025 using automatic dataloggers to provide 'continuous' readings at hourly intervals. The results of this monitoring are presented in Appendix C, which also displays recorded rainfall data[5] measured in close proximity to the site over the same period. Further monitoring is currently being undertaken at the time of writing this report to track seasonal variations in groundwater level over a longer period.

Table 2.4 summarises the piezometer screen levels and the highest / lowest recorded readings over the June 2025 monitoring period.

Table 2.4: Summary of recent groundwater monitoring

Piezometer ID		Screened zone (m RL)	Groundwater head (m RL)		Groundwater pressure (kPa)	
			Minimum	Maximum	Minimum	Maximum
BH01	Shallow	33.3 to 35.3	34.8	35.3	14.8	19.8
	Deep	29.3 to 31.3	30.3	30.4	10.0	11.1
BH01A	Shallow	35.3 to 37.3	36.8	37.0	14.5	16.7
	Deep	31.3 to 33.3	34.1		27.1	
BH02	Shallow	11.6 to 13.6	12.5	12.6	9.2	9.3
	Deep	3.1 to 5.1	5.7	5.9	25.5	27.5

Further groundwater monitoring data is also presented in the historical Aurecon reports near the centre of the site, which is summarised in Table 2.5 below.

[5] Rainfall data was retrieved from https://environmentauckland.org.nz/Data/

Table 2.5: Summary of historical groundwater monitoring

<table>
<tr><th>BH ID</th><th>Date range</th><th>Screened zone (m bgl)</th><th>Geological Unit</th><th>Groundwater head (m bgl) [RL m]</th><th>Groundwater pressure (kPa)</th></tr>
<tr><td>BH01</td><td>April 2018 to June 2018</td><td rowspan="4">3 to 6</td><td rowspan="4">Residual / Weathered ECBF</td><td>2.08 to 5.41 [39.6 to 42.9]</td><td>5.8 to 38.5</td></tr>
<tr><td>BH02</td><td>April 2018</td><td>2.32 to 2.85 [38.1 to 38.7]</td><td>30.9 to 36.1</td></tr>
<tr><td>BH05</td><td>April 2018 to March 2019</td><td>2.24 to 4.21 [28.82 to 31.75]</td><td>17.5 to 36.9</td></tr>
<tr><td>BH06</td><td>April 2018 to April 2019</td><td>4.41 to 5.26 [20.84 to 22.37]</td><td>7.3 to 15.6</td></tr>
<tr><td>BH42</td><td rowspan="6">February 2019 to April 2019</td><td>6 to 9</td><td>Weathered ECBF / ECBF rock</td><td>5.39 to 5.89 [27.15 to 27.65]</td><td>30.5 to 35.4</td></tr>
<tr><td>BH43</td><td rowspan="5">3 to 6</td><td rowspan="5">Residual / Weathered ECBF</td><td>2.72 to 2.89 [29.51 to 29.66]</td><td>30.5 to 32.2</td></tr>
<tr><td>BH44</td><td>3.87 to 4.44 [26.89 to 27.41]</td><td>15.3 to 20.9</td></tr>
<tr><td>BH46</td><td>2.75 to 3.70 [25.21 to 26.19]</td><td>22.6 to 31.9</td></tr>
<tr><td>BH48</td><td>3.38 to 3.86 [23.34 to 23.82]</td><td>21.0 to 25.7</td></tr>
<tr><td>BH50</td><td>3.19 to 3.57 [25.56 to 25.94]</td><td>23.8 to 27.6</td></tr>
</table>

The groundwater monitoring results to date confirm the following:

- A perched groundwater system has been detected within the upper ground profile, including within the shallow ECBF rock. The deeper regional groundwater table has not been intercepted by the groundwater monitoring in BH01 at the western end of the site and is inferred to be at a significantly lower elevation (estimated at between 7 mRL to 15 mRL).
- Non-hydrostatic pore water pressures are present, typically ranging between 10 and 30 kPa, indicating a transient (generally west to east and downwards) groundwater flow through the soil column.
- There is a slow to negligible response in perched groundwater level immediately following a rainfall event.
- Perched and regional groundwater levels fall from west to east. Perched groundwater levels fall from 37.0 mRL to RL 12.5 mRL across the site. Regional groundwater levels are inferred to be at approximately 6.0m RL at the eastern boundary rising towards the west.

3 Geotechnical considerations

The following geotechnical matters discussed within this report will need to be considered during detailed design of the proposed development:

- Seismic subsoil class and liquefaction
- Foundation design
- Groundwater drawdown effects, including subsoil drainage and settlement
- Retaining wall design
- Earthworks
- Slope Stability

Recommendations and opinions in this report are based on data from recent and historical site investigations undertaken at point locations. The nature and continuity of subsoil away from these locations are inferred, but it must be appreciated that actual conditions may vary from the assumed model.

There is nothing in the assessment of the following matters that constrain the proposed development from a geotechnical perspective that cannot be appropriately managed through conventional engineering controls.

3.1 Seismic considerations

3.1.1 Seismic site subsoil class

Seismic accelerations are dependent upon the stiffness of the underlying soil/rock. In terms of NZS 1170:2004[6], the site subsoil category for seismic design actions should be taken as **Category C** (shallow soil site).

3.1.2 Design accelerations

In terms of NZS 1170:2004 and MBIE[7], an importance level 2 structure has been assumed with the following seismic design events for a 50 year design life:

Table 3.1: Preliminary seismic design actions for design

Design Case	Return Period (yrs)	Peak Ground Acceleration	Earthquake Magnitude (M_w)
Serviceability Limit State (SLS)	1 in 25	0.05g	5.9
Ultimate Limit State (ULS)	1 in 500	0.19g	6.5

3.1.3 Liquefaction

The susceptibility of a site to liquefaction is a function of the soil plasticity, groundwater level, soil density, and level of earthquake shaking.

[6] NZS 1170.5:2004. Structural design actions. Part 5: Earthquake actions – New Zealand. SANZ.
[7] Ministry of Business, Innovation & Employment (November 2021). Earthquake geotechnical engineering practice MODULE 1: Overview of the Guidelines.

The near-surface material encountered at the site is predominantly silt/clay mixtures, typically highly plastic and cohesive in nature, overlying weakly cemented ECBF rock. Therefore, the risk of liquefaction following an ULS event (1 in 500 year) is considered negligible.

3.2 Foundation options

Based on the inferred ground conditions across the site, the following foundation options have been considered for the development:

Table 3.2: Recommended foundation options for development

Foundation Option	Considerations	Recommendation
Independent shallow pad or strip foundations	Unlikely to be feasible for the proposed 5 to 8 level structures due to insufficient resistance to uplift and overturning forces, particularly under seismic conditions.	**Not recommended.**
Shallow raft foundations bearing upon residual ECBF and/or weathered ECBF rock	Likely to be the most cost-effective foundation solution. Considered viable for the proposed building heights, particularly where full or partial basement levels are provided (i.e. fully compensated foundations). Settlement reduction piles or ground improvements could be considered, if necessary. Piles may be necessary for uplift loads or concentrated downward loads.	**Recommended for further assessment.**
Driven steel universal columns (UC) piles	Use is typically constrained by strict noise and vibration limits in residential environments, and therefore, unlikely to be a viable solution for this site.	**Not recommended.**
Screw piles	Screw piles enable rapid installation within the building footprint; however, they are typically limited in lateral load capacity, and may not be suitable for medium to high-rise buildings under seismic conditions.	**Potentially viable for further consideration**
Driven timber piles	Unlikely to provide sufficient axial capacity to support the proposed building. Pile groups would be required beneath each column, reducing cost-effectiveness. May be suitable as settlement-reduction piles beneath slabs, if required.	**Not recommended.**
Bored reinforced concrete piles embedded within ECBF rock	Likely to be a suitable foundation option for the proposed development where a raft is not feasible (e.g. concentrated loads or uplift loads). Bored piles can achieve high axial and lateral capacities, particularly when socketed into ECBF rock, and are well-suited to seismic loading conditions.	**Recommended for further assessment.**

The following outlines preliminary recommendations for the shallow raft foundation and bored pile options outlined above, which are preferred based on the preliminary development plans for the site.

3.2.1 Shallow raft foundations

A raft foundation is suitable for the proposed 5 to 7 level structures based on the site conditions observed. This option will require detailed consideration of differential settlement across the foundation due to both the differing "preconsolidation" (i.e. excavation depths) and soil stiffness at formation level across the building footprint.

Settlement reducing piles, such as driven timber piles or shallow ground improvement (e.g. RAPs), can be utilised to control these variable stiffness effects on the downslope end of the basement. This can be considered further during detailed design.

Preliminary design parameters are presented below suitable for concept design of the foundation. Given the variation in geological conditions and excavation depths across the proposed development area, a preliminary assessment has been undertaken for each proposed building, as presented in Figure 3.1 below.

Detailed settlement analysis will be required to be carried out during detailed design in co-ordination with the structural to confirm total / differential settlements are within acceptable tolerances. This is standard practice.

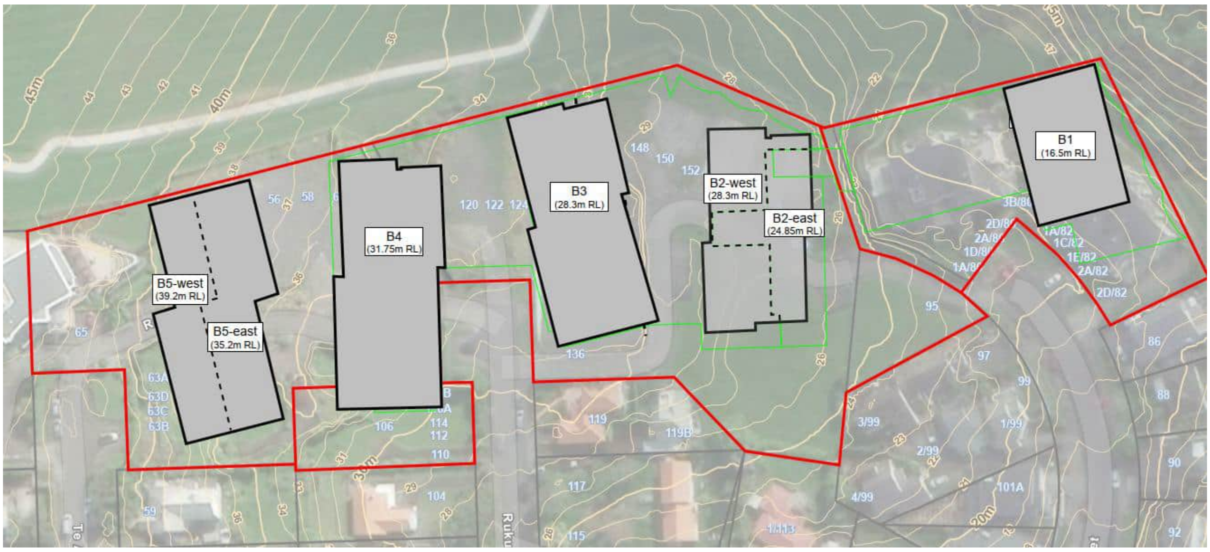


Figure 3.1: Building layout adopted for bearing capacity assessment

Table 3.3: Preliminary design parameters for shallow raft foundations

Zone	Floor level RL (m)	Min / Max Cut / Fill Depths	Foundation subgrade	Design Bearing Capacity (kPa)		Vertical spring stiffness (kPa/mm)*
				Geotechnical Ultimate	Ultimate Limit State (ϕ_g = 0.5)	
B5 west	39.2	0.8 m (cut) 4.2 m (fill)	Engineered Fill / Residual ECBF**	450	225	3 – 5
B5 east	35.2	3.6 m (cut) 0.2 m (fill)	Residual ECBF	450	225	3 – 20
B4	31.75	4.1 m (cut) 1.75 m (fill)	Engineered Fill / Residual ECBF	450	225	3 – 45

Zone	Floor level RL (m)	Min / Max Cut / Fill Depths	Foundation subgrade	Design Bearing Capacity (kPa)		Vertical spring stiffness (kPa/mm)*
				Geotechnical Ultimate	Ultimate Limit State (ϕ_g = 0.5)	
B3	28.3	5.2 m (cut) 0.7 m (cut)	Residual ECBF	450	225	2 – 65
B2 west	28.3	0.2 m (cut) 1.8 m (fill)	Engineered Fill / Residual ECBF**	450	225	2 -3
B2 east	23.4	4.6 m (cut) 3.2 m (cut)	Residual ECBF	450	225	5 – 10
B1	16.5	0.8 m (cut) 2.4 m (fill)	Engineered Fill / Residual ECBF**	450	225	3 – 10

*During structural design, variability in the calculated vertical soil spring stiffness should also be considered. We recommend that the designer checks the building response for a range of stiffness between 50% to 200% of the values provided.

** Parts of the foundation subgrade may comprise existing fill, which may need to be undercut and replaced during construction to utilise shallow raft foundations. Further investigations to further quantify the strength and thickness of the existing fill below the foundation are recommended, should shallow raft foundations be adopted in these areas.

3.2.2 Bored pile foundations

Bored piles founding within ECBF rock can be considered where shallow foundations are not determined to be (e.g. for concentrated or uplift loads).

The geotechnical parameters presented in Table 3.3 can be used for the design of bored piles. These properties have been assessed based on published correlations with insitu strength tests and our experience with similar geological units.

An ECBF rock contour plan with inferred rock levels is included in Appendix A to assist with future high-level estimation of building pile lengths. ECBF rock contours are inferred from ground investigations undertaken at point locations; therefore, some variations in ECBF rock level may occur. Should bored pile foundations be adopted, additional proof drilling across the building footprints is recommended immediately prior to construction to provide increased certainty on pile lengths before construction.

Table 3.4: Preliminary assessment for bored RC pile capacities founding in ECBF rock

Geological Unit	Shaft Capacity (2)		End-bearing Capacity (3)	
	Geotechnical Ultimate Capacity	Factored ULS Capacity (1) (ϕ = 0.5)	Geotechnical Ultimate Capacity	Factored ULS Capacity (1) (ϕ = 0.5)
ECBF rock	500 (smooth) 700 (grooved)	250 (smooth) 350 (grooved)	6,000	3,000

1 Strength reduction factor (ϕ_g) of 0.5 has been calculated based upon guidance in AS2159 for the ULS gravity and seismic case. If required, a seismic overstrength reduction factor of 0.7 can be applied to the geotechnical ultimate capacity values presented above, subject to the assessed vertical deformation of the pile being structurally satisfactory.

2 When assessing pile uplift capacity in non-grooved piles, a further reduction factor of 0.67 should be applied to the presented skin friction values.

3 Requires an embedment of at least 3 pile diameters into geological unit to mobilise end bearing capacity, otherwise end bearing can be reduced to ⅔ of recommended values.

Bored piles are generally unlikely to require significant temporary casing during excavation (other than temporary casing for safety, or to support any loose existing fill that may be encountered). However, some loose sand lenses were inferred within the residual ECBF soils across parts of the site, typically between 3 to 4 m bgl, which may require temporary support if encountered. This may be achieved with the use of either a temporary casing or the use of a drilling fluid (bentonite/polymer).

3.3 Groundwater effects

3.3.1 Groundwater levels

The groundwater monitoring completed to date (as presented in Section 2.7) indicates the presence of a perched groundwater regime within the soil profile, with the inferred regional groundwater level encountered within the ECBF rock at depth.

ECBF typically comprises layers of sandstones, siltstones and mudstones of varying permeability. As surface water (from rain and other sources) soaks into the ground, it seeps faster through more permeable sandy soils or rock, and seeps along the top of less permeable aquicludes such as clays or mudstones. Perched water can build up head where it collects over aquicludes, driving further seepage along and eventually downwards through the soil and rock profile to deeper water tables. In our experience across the Auckland area, groundwater pressure does not increase linearly with depth below ground. Our experience is that groundwater pressures within these perched water tables typically build up to a maximum of 10 kPa to 30 kPa (depending primarily upon water infiltration and vertical permeability).

This non-hydrostatic behaviour has been observed during the monitoring of the standpipe piezometers recently installed at the site, as presented in Appendix C, which displays the lowest water pressures measured within each piezometer screen between 13 June and 27 June 2025.

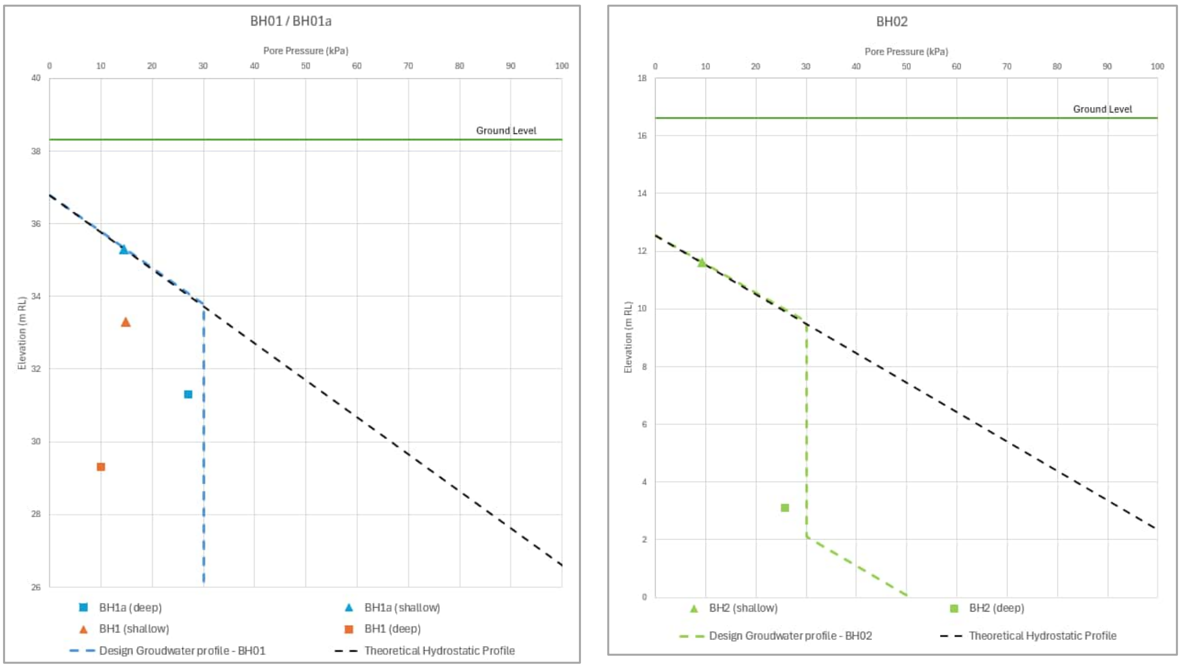


Figure 3.2: Groundwater pressure vs elevation plots from recent groundwater monitoring

As shown, the perched groundwater system present within the upper soil and weathered ECBF rock profile exhibits limited pore water pressures, typically in the range of 10 to 30 kPa, and is hydraulically disconnected from the deeper regional groundwater table.

Based on the monitoring undertaken, a design perched groundwater level has been adopted with a limiting pore water pressure of 30 kPa to represent the upper bound of observed transient perched conditions, as shown in Figure 3.1 above. Due to the shallow recorded depth of this perched groundwater level, it is expected to be encountered across large parts of the basement excavations.

The regional groundwater table was not encountered within the depth range of the installed monitoring wells, and is therefore considered that the regional groundwater table is located at a significantly lower elevation. As such, the regional groundwater level is considered unlikely to be intercepted during excavation or construction activities.

Due to the transient and variable nature of perched groundwater systems, it is likely that the groundwater pressures within these upper soil layers have fluctuated significantly over time, including periods where pressures were substantially lower than those currently observed. These fluctuations may have occurred naturally due to seasonal variations or during historical periods of drier conditions and limited recharge.

It is on the basis of the above investigations that the following analysis is undertaken.

3.3.2 Assessment against the Auckland Unitary Plan (operative in part)

We have reviewed the Auckland Unitary Plan (AUP) rules regarding the take, use, damming and diversion of groundwater that are relevant to the proposed basement excavations. The AUP provides for the take, using damming and diversion of groundwater and drilling in association with excavation. The AUP includes several requirements that must be met for permitted activities with regard to groundwater, which are listed in Activity Table E7.4.1 and Permitted Activity Standards E7.6.1.6 and E7.6.1.10.

The assessment of compliance with these rules is summarised below and identifies a number of non-compliances with the permitted standards. Our assessment of the project is that the resource consent is therefore required as a **restricted discretionary activity** in accordance with AUP, Table E7.4.1 (A20) and (A28), due to the following infringements:

- E7.6.1.6(2) – (Groundwater take in excess of 30 days).
- E7.6.1.6(3) – (Groundwater take beyond the construction period).
- E7.6.1.10(1)(d) – (Groundwater diversions exceeding 10 days).
- E7.6.1.10(2)(a) – (Site area exceeds 1 ha)
- E7.6.1.10(2)(b) – (Excavation below natural ground level exceeding 6 m).
- E7.6.1.10(3) – (Natural groundwater level must not be reduced by more than 2 m on the boundary of any adjoining site).
- E7.6.1.10(4)(b) – (Structures extending more than 2 m below natural groundwater level).

Table 3.5: AUP E7 engineering assessment

E7 Taking, using, damming and diversion of water and drilling - E7.6.1 PERMITTED ACTIVITY STANDARDS		
E.7.6.1.6 Dewatering or groundwater level control associated with a groundwater diversion permitted under Standard E7.6.1.10, all of the following must be met:		
Permitted Activity Standards:	**Geotechnical Interpretation of Compliance**	**Permitted activity compliance**
(1) The water take must not be geothermal water;	No geothermal water is expected to be present / encountered.	Yes

(2) The water take must not be for a period of more than 10 days where it occurs in peat soils, or 30 days in other types of soil or rock; and	The basement construction period is likely to extend beyond 30 days. The basement floor slab may be permanently drained, and therefore the water take will exceed 30 days.	No
(3) The water take must only occur during construction.	Basement floor slab may be permanently drained (i.e. not just during construction)	No
E7.6.1.10. Diversion of Groundwater Caused by any Excavation (including trench), or tunnel		
(A27) Permitted Activity Standards: Exemptions	**Geotechnical Interpretation of Compliance**	**Permitted activity compliance**
(1) All of the following activities are exempt from the Standards E7.6.1.10(2) – (6):		
(a) pipes cables or tunnels including associated structures which are drilled or thrust and are less than 1.2 m in external diameter;	There are not expected to be any pipes cables or tunnels ≥1.2 m.	N/A
(b) pipes including associated structures up to 1.5 m in external diameter where a closed face or earth pressure balanced machine is used;	N/A due to compliance with 1(a) above.	N/A
(c) piles up to 1.5 m in external diameter are exempt from these standards;	All piles are expected to be <1.5 m diameter.	Yes
(d) diversions for no longer than 10 days; or	Groundwater diversion will be longer than 10 days during construction	No
(e) diversions for network utilities and road network linear trenching activities that are progressively opened, closed and stabilised where the part of the trench that is open at any given time is no longer than 10 days.	The groundwater diversion is for purposes other than network utilities and road network linear trenching activities.	N/A
(2) Any excavation that extends below natural groundwater level, must not exceed:		
(a) 1 ha in total area: and	(a) Earthworks exceeds 1 ha	No
(b) 6 m depth below the natural ground level	(b) The maximum excavation depths will be greater than 6 m below existing ground levels (up to 8.75 m)	No
(3) The natural groundwater level must not be reduced by more than 2 m on the boundary of any adjoining site.	(3) The perched groundwater level is expected to be lowered greater than 2 m (up to 4.7 m) at the northern boundary.	No
(4) Any structure, excluding sheet piling that remains in place for no more than 30 days, that physically impedes the flow of groundwater through the site must not:		
(a) impede the flow of groundwater over a length of more than 20 m; and	(a) Permanent drained basements will not impede groundwater flow long-term	Yes
(b) extend more than 2 m below the natural groundwater level.	(b) The structure will extend more than 2 m below pre-development levels	No
(5) The distance to any existing building or structure (excluding timber fences and small structures on the boundary) on an adjoining site from the edge of any:		
(a) trench or open excavation that extends below natural groundwater level must be at least equal to the depth of the excavation;	(a) Adjoining buildings (primarily to the south of the development) are well off-set from the deepest basement excavation. Only minor excavations are required in proximity to these structures.	Yes

(b) tunnel or pipe with an external diameter of 0.2 - 1.5 m that extends below natural groundwater level must be offset 2 m or greater; or	(b) Any site connections are likely to be greater than 2 m from neighbouring structures.	Yes
(c) a tunnel or pipe with an external diameter of up to 0.2 m that extends below natural groundwater level has no separation requirement.	(c) No comment required	Yes
(6) The distance from the edge of any excavation that extends below natural groundwater level, must not be less than:		
(a) 50 m from the Wetland Management Areas Overlay;	The proposed excavation is not located within 50 m of a Wetland Management Area	Yes
(b) 10 m from a scheduled Historic Heritage Overlay; or	No Historic Heritage overlays are noted on Auckland Council GeoMaps	Yes
(c) 10 m from a lawful groundwater take	We are not aware of any lawful groundwater takes located within 10m of the excavation	Yes

- The relevant matters of discretion and assessment criteria relating to Rules E7.4.1 (A20) and (A28) are contained in Section E7.8.1(4) and (6), and E8.8.2(1), (6), and (10) of the AUP

3.3.3 Monitoring and Contingency Plan (M&CP)

A draft Monitoring & Contingency Plan has been prepared to support the Groundwater Take and Divert Consent application. This is presented in Appendix E**.**

Monitoring beyond the application site will include (but not be limited to):

- Ground settlement pins will be installed surrounding the basement excavations.
- Building pins will be installed for nearby structures.
- Building conditions surveys will also be undertaken on properties beyond the anticipated extent of effects of the development, subject to the landowner acceptance.

A final Groundwater and Settlement Monitoring and Contingency Plan (GSMCP) will be submitted to Council for certification prior to the commencement of dewatering. A condition of consent is proposed in this regard and included in Appendix F.

3.3.4 Groundwater drawdown settlement

Dewatering of the site (and beyond) has the potential to cause settlement of the surrounding ground due to the additional pressure applied to the soil skeleton as groundwater is removed.

Due to the transient and variable nature of perched groundwater systems, the groundwater pressures within these upper soil layers are expected to have fluctuated significantly over time, including periods where pressures were substantially lower than those currently observed. These fluctuations are likely to have occurred naturally due to seasonal variations or during historical periods of drier conditions and limited recharge. As a result, the soils underlying the perched system are expected to have already experienced any settlement associated with shallow groundwater drawdown.

We have undertaken an assessment of groundwater drawdown induced settlement based on the Theim-Dupuit Equation in accordance with Marinelli & Niccoli[8] to assess the effects of the proposed basement excavations. Analyses have been undertaken for the anticipated "worst case" groundwater drawdown for each of the proposed basements based on the current groundwater monitoring data, as summarised below:

Table 3.6: Groundwater drawdown depths for the proposed basement excavation

Building	Maximum Excavation Depth (m bgl)	Groundwater Level (m bgl)	Maximum drawdown (m)
1	4.00	2.50	1.50
2	8.75	3.80	4.95
3	5.90	4.40	1.50
4	5.45	2.90	2.55
5	4.80	2.20	2.60

The following soil parameters have been adopted in our drawdown analysis and subsequent assessment of one-dimensional consolidation settlement. These parameters have been derived from the results of our geotechnical investigations and our experience with similar materials.

Table 3.7: Groundwater drawdown and one-dimensional consolidation parameters

Geological Unit	Horizontal permeability, k_h (m/s)	Coefficient of volume compressibility, m_v (m^2/MN)
Residual ECBF Soils	1×10^{-7}	0.05
Weathered ECBF	3×10^{-7}	0.02
ECBF Rock	3×10^{-7}	Negligible settlement

A one-dimensional consolidation analysis has been undertaken to assess the potential settlement as a result of dewatering based on the increases in total stress as a result of the estimated groundwater drawdown. The settlement is estimated based on the following:

$$Settlement = \Delta\sigma_v \times m_v \times H$$

Where:

- $\Delta\sigma_v$ =change in effective stress due to groundwater drawdown
- m_v = coefficient on volume compressibility as presented in Table 3.4
- H = thickness of layer

Non-hydrostatic groundwater pressure limited to a maximum of 30 kPa has been assumed in the settlement analysis for the perched groundwater.

The magnitude of groundwater drawdown and consolidation settlement decreases extending away from the excavation. The estimated drawdown settlement at different offsets are presented in Table 3.8.

[8] Fred Marinelli and Walter L. Niccoli (March-April, 2000). Groundwater, Vol. 38, No. 2, Simple Analytical Equations for Estimating Ground Water Inflow to a Mine Pit.

Table 3.8: Groundwater Drawdown at different offset from the excavation

Building 1	Offset from the edge of the excavation (m)	0	0.8	3.1	6.2	8
	Calculated Drawdown (m)	1.5	0.8	0.3	0.1	0.0
	Calculated Settlement (mm)	4.0	2.0	1	0.5	0
Building 2	Offset from the edge of the excavation (m)	0	1.2	4.6	6.9	11.5
	Calculated Drawdown (m)	5	2.7	1.1	0.7	0.0
	Calculated Settlement (mm)	5.0	4.5	2.0	0.5	0.0
Building 3&4	Offset from the edge of the excavation (m)	0	0.4	1.5	3.0	3.7
	Calculated Drawdown (m)	1.6	0.9	0.4	0.1	0
	Calculated Settlement (mm)	3.0	1.5	0.5	0.1	0.0
Building 5	Offset from the edge of the excavation (m)	0	0.2	1.0	2.0	2.5
	Calculated Drawdown (m)	2.6	1.4	0.6	0.2	0
	Calculated Settlement (mm)	3	1.5	0.5	0.1	0
Presented groundwater drawdown and settlement are presented to two significant figures for comparison only. The precision of the results are closer to a single significant figure of accuracy.						

3.3.5 Permanent Basement Considerations

As discussed above, the potential settlement as a result of the drawdown of the perched groundwater is predicted to result in negligible settlements. As such, the groundwater-associated settlements are likely to have less than minor effects on neighbouring structures. On this basis, a fully drained basement floor is considered to be a suitable option. The drained basement floor system is recommended to comprise the following:

1 To prevent hydrostatic uplift pressures on the underside of the basement slab, an underdrainage system should be installed discharging via gravity or pumping to a reticulated storm water system. It is envisaged the system would comprise the following:
   - (i) Minimum of 200 mm thick permeable hardfill layer across the basement slab; e.g. GAP65).
   - (ii) 200 mm deep by 200 mm wide subsoil drains with perforated drain pipes (110 mm dia Nexus Hi-way) installed at approximately 6 m centres west-east beneath the basement slab, with inspection points at both the ends of each subsoil pipe. The drains should be backfilled with well graded aggregate (e.g. GAP65) and have a flushing port at each end.
   - (iii) A nonperforated subsoil collector drain installed inside the perimeter of the retaining walls to collect groundwater flow and re-direct to a discharge point.

As a failure of the drainage system could result in groundwater pressures over-stressing the basement slab, the design of the drainage should consider robustness, regular maintenance/monitoring, and contingencies against failures of drainage system components.

If a drained basement floor is utilised, consistent with the above no significant groundwater mounding is expected to occur upslope (west) of the basement walls.

Conditions of consent are proposed to reflect this and are included in Appendix F.

3.4 Ground retention

Architectural drawings[2] for the development identify that each of the proposed buildings will include of a basement of 1 to 2 levels, with Buildings 2, 3 & 4 including a shared basement that will extend between the across the above ground building footprints.

The proposed basement levels, and the excavation required to form these basements are outlined in Table 3.9 below.

Table 3.9: Summary of excavation levels

Building	Proposed basement RL (m)	Maximum temporary excavation height (m bgl)
1	16.5	4.00
2	19.95 to 23.4	8.75
3	28.3 to 31.75	5.90
4	31.75	5.45
5	35.2	4.80

Depths are based on LiDAR survey available and is indicative only

Temporary retention for the basement will be required along much of the building perimeter. The following will need to be considered in the detailed design for selection of the method of retention:

- Proximity to neighbouring buildings, services and/or property boundaries – temporary retention will be required to maintain stability and minimise effects where excavating in proximity to neighbouring buildings / services. Proximity may require construction of temporary or permanent retaining walls to control ground movements.
- Walls should be sufficiently stiff to control ground deformations and potential settlement effects. This may require installation of temporary props or anchors for walls of greater height and/or walls of higher stiffness (i.e. concrete not timber).
- Utilise a construction technique that will not cause significant vibrations or noise to adjacent property users. A bored concrete or timber wall is considered more suitable than a driven pile wall in close proximity to residential structures.

Based on the above considerations, the following retention options are considered viable for the site:

1. **Temporary batter slopes** – a large portion of the basement excavations will be well within the site boundaries, which will allow sufficient space to enable temporary batters, which are likely to be the most cost-effective solution to support the basement excavation. Using the soil parameters presented in Section 3.4.1, simplified stability charts[9] and a minimum temporary factor of safety of 1.3, the following temporary batters may be formed (Table 3.10).

   The cut slopes will require inspection by a suitably qualified Geotechnical Engineer to confirm that the proposed slopes are compatible with the actual ground conditions encountered.

   Construction surcharges should be maintained a minimum of 2 m offset from the slope crest, with measures put in place to keep the cuts dry (polythene matting or similar).

[9] Wyllie, C. and Mah, W. (2004) Rock Slope Engineering Civil and Mining. In: Hoek, E. and Bray, J.W., Eds., Rock slope Engineering, Taylor & Francis Group, London and New York, 431 p. Adopting Chart 2 of 5.

Table 3.10: Preliminary temporary slope batter options

Ground Conditions	Max. Batter Height	Batter Slope
Non-engineered fill	Up to 3 m	30 degrees
Residual & Weathered ECBF	Up to 5 m	35 degrees

Temporary batters greater than 5 m in height should be subject to specific design.

2 **Temporary RC bored soldier pile retaining walls –** likely required where the basement extends near to neighbouring buildings, services and/or boundaries. The following sections outline the preliminary design of the temporary retaining walls. <u>This wall type has been adopted for the preliminary design of the retention.</u>

3 **Driven sheetpile retaining walls –** may be considered along the north extent of stage 2 and stage 4 and west extent of stage 4, where the site footprint is away from the nearby residential buildings, and the noise / vibrations limits may not be exceeded. This will be subject to a detailed assessment by an Acoustic specialist consultant.

A conceptual extent and methodology for temporary retention of the basement excavations are presented in Appendix D. Temporary retaining walls will be required where excavations occur close to site boundaries, particularly along the northern boundary. For other areas of the basement, both temporary battered slopes and temporary retaining walls are considered viable, subject to local geometry and construction staging.

It is recommended that optioneering be undertaken during detailed design to confirm the preferred forms of temporary retention. This should consider the relative cost-effectiveness of each option, along with excavation access requirements and site-specific constraints. Early contractor input at this stage of design would be valuable to assess and refine these options.

Permanent retention for the basement walls is proposed to be provided by pre-cast/insitu concrete walls designed by the structural engineer.

3.4.1 Geotechnical parameters

The geotechnical parameters presented in Table 3.11 are recommended for retaining wall assessment.

Table 3.11: Summary of geotechnical parameters

Parameters		Units	Existing Fill	Residual ECBF	Weathered ECBF	ECBF Rock
Unit Weight	γ	kN/m³	18	18	19	20
Effective Cohesion	c′	kPa	3	5	10	50
Friction Angle	ϕ'	Deg	28	30	32	40
Youngs Modulus	E	MPa	15	25	50	400
At rest earth pressure	Ko	-	0.53	0.50	0.47	0.36
Active earth pressure	Ka	-	0.31	0.28	0.26	0.18
Passive earth pressure	Kp	-	3.81	4.29	4.8	8.38
Poisson's ratio	v′	-	0.3	0.3	0.3	0.25

The geological model has been based on borehole observations at point locations. The nature and continuity of geological conditions away from these locations are inferred, and actual conditions may vary from the assumed model.

- The geotechnical analysis of the retaining walls has been undertaken using pseudo non-linear finite element software[10] with the following design assumptions:
- Factor of Safety (Burland Potts) for overall wall stability ≥1.5 (temporary and seismic stability).
- To calculate the active and passive pressure coefficient, the wall friction was assumed to be 2/3 of the soil friction angle on the active side, and 1/2 of the soil friction angle on the passive side of the wall.

3.4.2 Structural parameters

Three representative sections have been selected for retaining wall analysis, as shown in Figure 3.3 below. These sections were selected to represent "worst case" sections for various retained heights, based on ground conditions and proximity to neighbouring structures (where applicable).

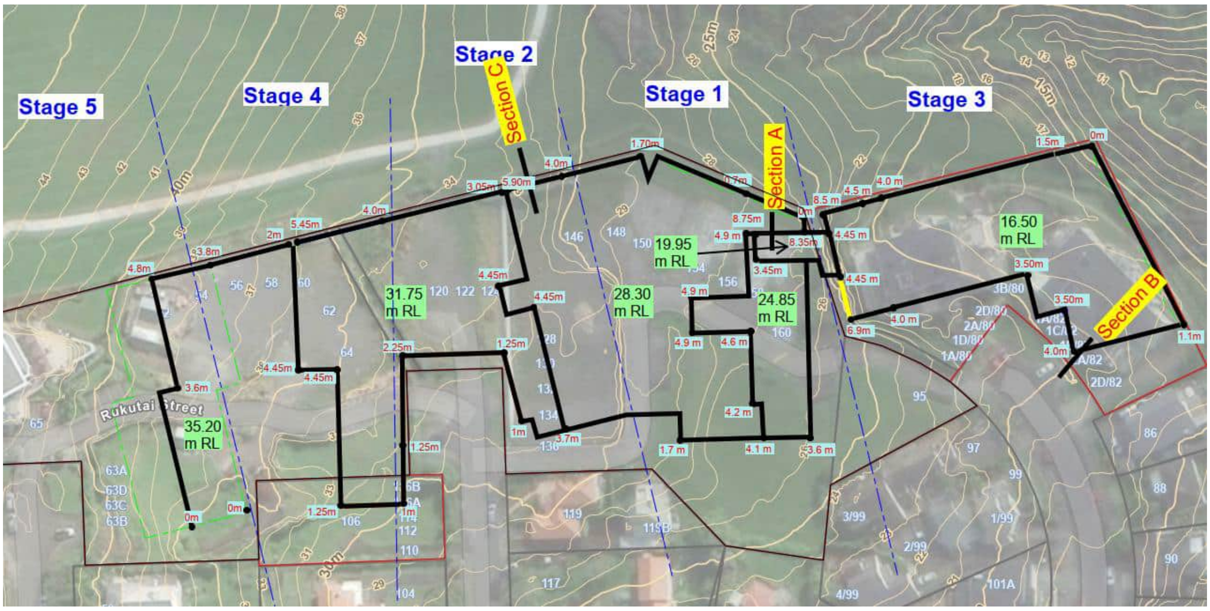


Figure 3.3: Analyses section locations adopted for preliminary design of temporary retaining walls

The following structural properties have been adopted for the preliminary design of the retaining walls. Structural design of the retaining walls is to be undertaken by others.

[10] Geosolve (2013). WALLAP. Anchored and Cantilevered Retaining Wall Analysis Program. Version 6.05.

Table 3.12: Structural properties

Sections	Section A	Section B	Section C
Retaining Wall ID	**Type 3**	**Type 2**	**Type 1**
Pile diameter (m)	0.75	0.60	0.75
Pile spacing (m)	2.25	1.8	2.25
Maximum retained height (m)	8.75	4.0	5.9
Pile Length (m)	12.35	10	12.2
Young's Modulus (GPa)	27	27	27
Pile second moment of area – gross section Ig (m4/)	6.9×10^{-3}	3.5×10^{-3}	6.9×10^{-3}
Prop Details			
Size	250UC90	-	-
Maximum prop length (m)	6.5	-	-
Prop spacing (m)	5.0		
Cross-sectional area (m^2)	0.0144	-	-

3.4.3 Surcharge loading

Based on the selected basement retaining wall sections, surcharges behind the proposed walls are summarised in Table 3.13 below.

Table 3.13: Summary of surcharges

Section	Adjacent Structures	Surcharges (kPa)	Offset to the wall (m)	Length of surcharge (m)
A to C	Construction traffic loading	12	2	10

3.4.4 Retaining wall analyses

Retaining wall analyses results are presented in Table 3.14. A summary of the assumed construction sequence/methodology is presented below:

<u>Section A – Propped Retaining Wall</u>

1 Install wall and capping beam from existing ground level;
2 Apply construction traffic surcharge loading;
3 Reduce wall stiffness to 75% cracked section stiffness (temporary case);
4 Excavate to 1.0 m below existing ground level for prop installation;
5 Install temporary prop to the capping beam at 0.5 m below existing ground level; and
6 Complete full excavation to bulk excavation level.

<u>Section B & C – Cantilever Retaining Walls</u>

1 Install wall and capping beam from existing ground level;
2 Apply construction traffic surcharge loading;
3 Reduce wall stiffness to 75% cracked section stiffness (temporary case); and

4 Undertake excavation to bulk excavation level.

Table 3.14: Summary of retaining wall analyses

	Output	Section A RTW Type 3	Section B RTW Type 2	Section C RTW Type 1
	WALLAP file	Section_A - 750RC@2.25m_cs_250UC90_prop.dat	Section_B - 600RC@1.8m_cs_Cantilever.dat	Section_C - 750RC @ 2.25m cs_Cantilever.dat
Wall Geometry	Pile Diameter (m)	0.75	0.60	0.75
	Pile Spacing (m)	2.25	1.8	2.25
	Pile Length below existing ground level (m)	12.35	10	12.2
	Max Temporary Retention Height (m)	8.75	4.0	5.9
Analysis results (temporary wall)	Max horizontal displacement (mm)	18	23	53
	Depth of max. displacement (m)	4.5	0 (ground surface)	0 (ground surface)
	Factor of Safety	2.0	1.6	1.6
	Max Bending Moment (kNm/m)	215	65	200
	Max Shear Force (kN/m)	150	35	85
	Prop Load (kN/prop)	450	-	-

Design loads presented above are unfactored. Structural design of the walls should include appropriate load factors where appropriate.

In accordance with the retaining wall analysis results above, a conceptual temporary basement retention system has been prepared and presented in Appendix D.

3.5 Assessment of combined effects

3.5.1 Estimated combined ground settlement

Ground surface settlements will result from a combination of the mechanical (wall deflections) and consolidation (groundwater drawdown) settlement effects, where applicable. The maximum total settlement will be in the long term and is a superposition of the settlement values from each source.

Combined ground settlement contours extending away from the basement excavation have been estimated based on the analyses outlined in the preceding sections, presented in Appendix A. As these assessments have been undertaken at localised “worst case” locations, interpolation has been applied to estimate settlements in areas where excavation depths and associated settlements reduce progressively. These interpolated values provide a broader representation of expected ground movements surrounding the site and have been utilised to assess potential effects on the surrounding environment, as outlined in the following sections.

Calculated ground movements surrounding the proposed excavations are shown on Figure 4 in Appendix A.

3.5.2 Effects on neighbouring buildings

Ground deformations associated with basement excavation have the potential to affect adjacent buildings. The tolerance of a building to vertical and horizontal ground movements depends on factors such as the type of construction, foundation system, structural redundancy, and the existing condition of the building.

Neighbouring buildings have been assessed to estimate potential effects due to development in accordance CIRIA (1996)[11] guidance as presented in Table 3.15. The building locations are shown on Figure 5 in Appendix A.

Table 3.15: Typical values of maximum settlements for building damage risk assessment

Risk Category	Maximum Differential Settlement	Maximum Settlement of Building (mm)	Description of Risk
1	Less than 1 in 500	Less than 10	Negligible: superficial damage unlikely
2	1 in 500 to 1 in 200	10 to 50	Slight: possible superficial damage which is unlikely to have structural significance
3	1 in 200 to 1 in 50	50 to 75	Moderate: expected superficial damage and possible structural damage to building, possible damage to relatively rigid pipelines
4	Greater than 1 in 50	Greater than 75	High: expected structural damage to buildings and rigid pipelines or possible damage to other pipelines

Table 3.16 below summarises the total and differential settlements estimated across neighbouring structures based on the analysis undertaken. This indicates the proposed basement excavation is likely to have negligible effects (marginally “slight”) on the neighbouring structures.

Table 3.16: Typical values of maximum settlements for building damage risk assessment

Building	Building description	Basement Excavation Depth (m)	Offset (m)	Likely retention method	Estimated settlement (mm)		Risk Rating
					Total	Differential	
104 Rukutai Street	Residential Dwelling	1.25	18.0	Temporary batter or Type 2 RTW	<5	Less than 1 in 1,000	Negligible
119 Rukutai Street	Residential Dwelling	3.70	10.0	Temporary batter or Type 2 RTW	<5	Less than 1 in 1,000	Negligible
119B Rukutai Street	Residential Dwelling	3.70	16.0	Temporary batter or Type 2 RTW	<5	Less than 1 in 1,000	Negligible
86 Aotea Street	Residential Dwelling	4.00	14.5	Temporary batter or Type 2 RTW	<5	Less than 1 in 1,000	Negligible

[11] CIRIA (1996). Project Report 30: Prediction and effects of ground movements caused by tunnelling in soft ground beneath urban areas.

The above methodology adopts a conservative approach by modelling the building as a weightless, linear elastic, isotropic beam, and by assuming that the presence of the building does not influence the surrounding "greenfield" ground displacement profile. In reality, the structural stiffness and load distribution of a building typically interact with the underlying ground, moderating deformations. If the building stiffness were explicitly accounted for, the resulting total and differential settlements experienced by the structure would be expected to be significantly lower than those estimated using the current approach.

3.5.3 Effects on utilities

While many types of utilities can accommodate high levels of differential settlement, certain types can be susceptible to damage. In general, a utility's tolerance to settlement depends upon the construction type/material, existing condition, and whether the utility runs parallel or perpendicular to the excavation. Utilities running perpendicular to the excavation works are considered to be at the highest risk of damage. Utilities which run parallel and are near the excavation works may experience horizontal displacement associated with ground loss at the excavation face; however, they will experience a much gentler differential settlement.

The methodology to assess the effects on utilities is based on the method on O'Rourke and Trautman (1982)[12] – which provides guidance on allowable differential settlement for various utility construction types, as presented in Table 3.17 below.

Table 3.17: Allowable differential settlement based on utility type

Utility type	Maximum allowable differential settlement (V:H)
Brick unlined	1:245
Welded steel pipe	1:122
Cast in-situ concrete	1:173
PVC & HDPE	1:67
Reinforced concrete pipe	1:229
Ductile iron pipe	1:229
Vitrified clay pipe	1:299
Cast iron pipe	1:150 – 1:500 (varies based on diameter)

Table 3.18: below presents a summary of the estimated total and differential settlements beneath services surrounding the development. Several services within and surrounding the site are understood to become redundant or be replaced as part of the development, and therefore no specific assessment has been undertaken.

Settlement contour profiles with distance perpendicular to the excavation are presented in Figure 4 in Appendix A. Based on this assessment, estimated settlements from the proposed development are within acceptable limits. The calculated settlement along each potentially affected service are presented in Table 3.18 below. The project Civil Engineer should confirm that the estimated settlement for each service is acceptable.

[12] O'Rourke, T D, and C H Trautmann. 1982. Buried pipeline response to tunnel ground movements. In Europipe 82 Conf., Basel, Switzerland, paper 1.

Table 3.18: Estimated settlement below underground services

Street	Services within pavement	Service orientation to excavation	Estimated ground settlement below service (mm)	
			Total	Differential (1)
Michael Joseph Savage Memorial Park	Stormwater line – 450 Concrete	Perpendicular	5 to 15	1:450
Michael Joseph Savage Memorial Park	Stormwater line – 375 Concrete	Perpendicular	5 to 15	1:450
Aotea Street	Stormwater line – 225 Concrete	Diagonal	5 to 10	1:750
Rukutai Street	Stormwater line – 300 Concrete	Diagonal	5 to 10	1:750
Rukutai Street	Wastewater line – 150 PVC	Parallel	< 5	Negligible

(1) Differential settlements estimated for services which are orientated perpendicular to the excavation only. Where the services run parallel to the excavation face, negligible differential settlements are expected to occur.

3.5.4 Effects on public roads

Excavations alongside roads have the potential to impact local roads. Settlement criteria for roads owned by Auckland Transport are not readily available.

For the purposes of this assessment, it has been assumed that the settlement tolerance limits for local road will be as follows (based on our experience on other project interfaces with Auckland Transport):

- 20 mm total vertical displacement
- 1 in 500 differential settlement

Based on our settlement analyses undertaken, the following impacts have been estimated on the surrounding roads:

Table 3.19: Estimated settlement below public roads

Road name	Offset (m)	Estimated settlement (mm)	
		Total	Differential
Aotea Street	8.5	<5	Less than 1 in 500

It is considered likely that the surrounding footpaths and roads will be upgraded as part of the works to interface with the proposed development. As such, any long-term settlement effects in these areas are expected to be negligible.

3.6 Floor slab and pavements

The subgrade conditions for the floor slab on grade and external pavements are expected to vary across the site and between excavation levels. A preliminary design subgrade CBR of 3% may be adopted for the design of a floor slab on grade and external pavements.

The exposed subgrade will be proof rolled with a roller with a static weight of at least 6 tonnes to identify any soft areas during construction. Granular material will be placed beneath the slab or external pavements with a minimum thickness of 150 mm to provide drainage and effectively distribute pressures onto the underlying material.

3.7 Earthworks considerations

Based on the preliminary development plans[2], a significant volume of earthworks will be required to form the finished levels across the site. This includes fill of up to 3.5 m thickness in the eastern parts of the site between Buildings 1 & 2.

Fill required to raise the site levels is proposed to be placed in accordance with an engineering specification, developed following selection of a fill source and the recommendations of NZS 4404:2010 – Land Development and Subdivision Infrastructure, and subject to confirmatory engineering inspections and in-situ testing.

The site will be stripped of any hardstanding and unsuitable materials (i.e. topsoil) prior to placement of any fill.

Areas of significant fill can result in consolidation settlement occurring within the underlying soils. This will be assessed during detailed design, with allowance for this settlement to occur prior to construction of the overlying buildings and/or pavements made within the construction programme where significant settlements are estimated.

Site-won material at the site from areas of cut is likely to comprise existing fill and residual ECBF soils comprising predominantly of cohesive silt / clay mixtures. This material is considered likely to be suitable for re-use at the site subject to review on site by a suitably qualified Geotechnical Engineer. Careful handling and conditioning of this material will be required to maintain suitability for re-use. This will be outlined within the engineered fill specification prepared during detailed design.

Material stockpiles at the site shall be located to avoid surcharging the adjacent temporary batters and retaining walls.

Based on the inferred ground conditions at the site, excavations for the basement are predicted to be predominantly within stiff very stiff soils that can be excavated using conventional earthmoving equipment and techniques.

Localised areas of the basement footprint may encounter ECBF rock within the excavation. This is predicted to be excavatable using larger conventional plant, such as a 20-tonne digger with rock teeth bucket or a ripping attachment. The ECBF rock generally has an unconfined compressive strength of 1 MPa to 5 MPa, with possible thin seams of cemented rock (i.e. Parnell grits) of up to 20 MPa strength.

3.8 Slope stability

3.8.1 AUP Considerations

The site is generally flat to gently sloping at an angle of less than 5-10 degrees falling from west to east.

A series of 30-40° benches are present near the centre of the site at the eastern edge of the former terraced housing (recently demolished). These are man-made cut / fill benches associated with the former development. Based on the proposed development plans, these benches will be excavated as part of the basement construction.

Beyond the site to the east, the ground profile falls at approximately 15-20° towards the inland coastal cliffs, which are present approximately 40 m north-west of the site boundary.

Based on the competent ground conditions encountered below the site, comprising very stiff cohesive deposits overlying ECBF rock, and the gently sloping ground in the proposed development area, slope stability risk to the proposed development is considered to be low.

The operative provisions of the AUP include a definition (and related provisions) of 'land that is potentially subject to land instability'. A review of the slope instability characteristics considered in Chapter E36 of the AUP-OIP has been undertaken for this assessment.

Table 3.20: AUP considerations – slope stability

Chapter E36 AUP Consideration	**Site Conditions**
Any land with one of the following characteristics:	
Where the land which is underlain by Allochthonous soils has slope angles greater than or equal to 1 vertical to 7 horizontal;	Allochthonous soils are not present at the site.
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;	Holocene or Pleistocene sediments have not been encountered at the site
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;	The ground profile at the site generally does not exceed 1(v):3(h), excluding the man-made benches outlined above (which will be removed as part of the development).

Further, PC120 proposes new provisions for landslide hazards, that have immediate legal effect.

A landslide hazard risk assessment has been prepared in accordance with Appendix 24 of the Auckland Unitary Plan[13], which sets out Auckland Council's prescribed methodology for assessing landslide hazard risk for land use and land use changes. This provides a quantitative and semi-quantitative basis for evaluating landslide risk significance within mapped susceptibility areas.

The assessment process follows four key stages:

1 **Stage 1 – Desk Study:** Identification of any evidence of historical or potential landslide hazards affecting the site based on the landslide hazard maps provided on Auckland Council GeoMaps[14].

2 **Stage 2 – Method Selection:** Determination of the appropriate risk assessment method (Method 1 or 2) based on mapped susceptibility and the sensitivity of the proposed land use.

3 **Stage 3 – Risk Assessment:** Application of the selected method to determine the likelihood, consequences, and overall risk classification associated with potential landslide events.

4 **Stage 4 – Planning Application:** Use of the assessed risk level to determine the appropriate activity status under Table E36.4.1B of the Auckland Unitary Plan

The following sections detail the landslide hazard risk assessment undertaken in accordance with the prescribed methodology.

3.8.1.1 Stage 1 – Desk Study

Landslide susceptibility in the Auckland Council study was assessed for two distinct landslide types:

- Shallow, smaller scale landslides; and
- Large-scale landslide features

Risk was assessed based on potential landslide-inducing variables, including local conditions, such as topography and geological conditions, and statistical analysis of a 'landslide inventory' of historical

[13] https://new.aucklandcouncil.govt.nz/en/plans-policies-bylaws-reports-projects/our-plans-strategies/unitary-plan/auckland-unitary-plan-modifications/proposed-plan-changes/pc-120-housing-intensification-resilience.html
[14] https://geomapspublic.aucklandcouncil.govt.nz/viewer/index.html

mapped landslides. The results were divided into 5 risk classes (Very Low, Low, Medium, High and Very High).

Based on a review of the landslide hazard desktop information provided on Auckland Council Geomaps, the following has been identified:

- The landslide inventory indicates **no historical landslides** have been mapped in the vicinity to the site, with the nearest landslide noted >1km away on Ngapipi Road to the south-west.
- **Shallow landslide risk** is generally 'very low to low' across the majority of the site consistent with our independent assessment, however 'very high' landslide risk is noted within the central and western parts of 80 – 82 Aotea Street at the eastern end of the development (refer Figure 3.4). This part of the site is currently occupied by multi-level buildings formed on a level building platform, with a boundary retaining wall along the western boundary.

  This is not consistent with the assessment guidance[15] provided, where 'Very high to High' susceptibility is primarily resulting from steep to very steep hillslopes, coastal cliffs and bluffs, and moderate to steep slopes underlain by weak or soft materials, with these areas having a high concentration of past slope instability.

  It is considered likely that the presence of the retaining wall and the apparent 'steep slopes' inferred from contour data across the retaining wall have overestimated the slope instability susceptibility in the modelled output for this portion of the site. Based on the site-specific observations and ground conditions from the recent site investigations, and the distribution of mapped susceptibility across the wider site, the **shallow landslide risk has been revised to 'Moderate'** for the purpose of this assessment.
- **Deep landslide risk is low** across the entirety of the site as shown in Figure 3.5.

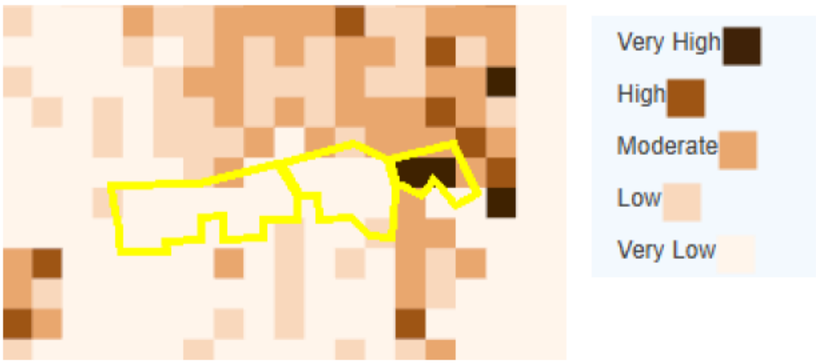


Figure 3.4: Shallow landslide susceptibility maps (Auckland Geomaps)

[15] WSP (28 February 2025). Report to Auckland Council. *Auckland Landslide Susceptibility Study. Technical Report 2025*. Ref: GS 2025/09.

Figure 3.5: Deep landslide susceptibility maps (Auckland Geomaps)

3.8.1.2 Stage 2 – Method Selection

The potential for landslide occurrence across the site was reviewed using Auckland Council's published Landslide Susceptibility and Inventory Maps. The mapped data indicates that the site exhibits very low to moderate susceptibility to shallow landslides, with no evidence of recent or active landsliding. Local terrain comprises gentle to moderately inclined slopes underlain by very stiff weathered soils overlying rock, with limited potential for large-scale slope instability.

In accordance with Stage 2 – Risk Assessment Method Selection of Appendix 24, areas with moderate susceptibility in combination with a medium to large scale development require assessment under **Method 1 – Semi-Quantitative Risk Assessment**. This approach is appropriate where the anticipated landslide risk significance is low to moderate, and where quantitative modelling is not warranted.

3.8.1.3 Stage 3 – Method 1 – Semi-Quantitative Risk Assessment

The landslide risk for the site has been assessed using Method 1 (Semi-Quantitative Assessment) as prescribed under Appendix 24 of the Auckland Unitary Plan.

This method requires identification of at least three landslide hazard scenarios that represent a 'high likelihood, 'median likelihood' and 'maximum credible event'. The following scenarios have been developed for consideration at the site:

- **High Likelihood (Frequent Event):**
  - A small, shallow debris slide or soil creep occurring on a steep section of the slope during intense rainfall.
  - Typically limited in area and depth (e.g. less than 1 m deep), affecting surface soils or fills.
- **Median Likelihood (Occasional Event):**
  - A moderate-sized rotational or translational slide triggered by prolonged wet weather.
  - May involve movement of colluvial or weathered materials, potentially affecting part of a foundation platform or access road.
- **Maximum Credible Event (Rare Event):**
  - A deep-seated or extensive shallow landslide involving major slope failure under extreme rainfall or seismic loading.
  - Could mobilise large volumes of material with potential runout to lower portions of the property or neighbouring sites.

Based on the Engineering Geological Model developed for the site as outlined in this report, an evaluation of the likelihood and consequence of each event considering the site-specific conditions has been undertaken in accordance with the below assessment tables provided by Appendix 24 of the Auckland Unitary Plan.

Table 3.21: Likelihood Categories for Landslide Hazard Scenarios

Likelihood Category	Likelihood Descriptor	Indicative Value of Approximate Annual Probability	Equivalent AEP
Almost certain	The event is expected to occur over the likely duration of the activity.	10^{-1} (1 in 10)	10 %

Likelihood Category	Likelihood Descriptor	Indicative Value of Approximate Annual Probability	Equivalent AEP
Likely	The event will probably occur under adverse conditions over the likely duration of the activity.	10^{-2} (1 in 100)	1 %
Possible	The event could occur under adverse conditions over the likely duration of the activity.	10^{-3} (1 in 1,000)	0.1 %
Unlikely	The event might occur under very adverse circumstances over the likely duration of the activity.	10^{-4} (1 in 10,000)	0.01 %
Rare	The event is conceivable but only under exceptional circumstances over the likely duration of the activity.	10^{-5} (1 in 100,000)	0.001 %
Barely credible	The event is inconceivable or fanciful over the likely duration of the activity.	10^{-6} (1 in 1,000,000)	0.0001 %

Table 3.22: Consequence Categories for Landslide Hazard Scenarios (less than 5 ha)

Consequence Category	Human Safety	Critical Buildings	Community Buildings	Buildings Accommodating Activities Sensitive or Potentially Sensitive to Natural Hazards *(excluding critical or community buildings)*
Catastrophic	> 10 dead and/or > 1000 injured	Building unusable for > 1 week	Building unusable for or more than 1 month	Structure(s) completely destroyed and/or large-scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property major consequence damage.
Major	1 to 10 dead and /or 101 – 1000 injured	Evacuation of building required and/or building unusable for 1 week or less	Building unusable for 1 week to 1 month	Extensive damage to most of structure and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.
Medium	0 dead, 11 – 100 injured	Building in landslide hazard risk assessment area but useability not affected	Evacuation of building required and/or building unusable for 1 week or less	Some damage to structure and/or requiring local stabilisation works. Could cause at least one adjacent property minor consequence damage.
Minor	0 dead, 1 – 10 injured	N/A	Building in landslide hazard risk assessment area but useability not affected	Minor damage to part of structure and/or part of site requiring some reinstatement or minor stabilisation works.
Insignificant	0 dead, 0 injured	Building outside landslide hazard risk assessm	Building outside landslide hazard risk as	Little damage.

Consequence Category	Human Safety	Critical Buildings	Community Buildings	Buildings Accommodating Activities Sensitive or Potentially Sensitive to Natural Hazards *(excluding critical or community buildings)*
		ent area and useability not affected	sessment area and useability not affected	

Table 3.23: Risk Classification Table

		Consequence category				
		Insignifcant	**Minor**	**Medium**	**Major**	**Catastrophic**
Likelihood Catgory	**Almost Certain**	Medium	High	High	High	High
	Likely	Low	Medium	High	High	High
	Possible	Low	Low	Medium	High	High
	Unlikely	Low	Low	Low	Medium	High
	Rare	Low	Low	Low	Low	Medium
	Barely credible	Low	Low	Low	Low	Low

The following criteria have been applied to the three landslide scernarios outlined above:

Table 3.24: Landslide scenario assessment

Landslide Scenario	Description	Likelihood	Consequence	Risk Category
High likelihood	Shallow debris slide during intense rainfall. Limited depth and footprint. Unlikely to pose any risk to life or the proposed buildings. Considered unlikely to occur based on the competent ground and moderate slopes surrounding the site.	Unlikely	Insignifcant	**Low**
Median likelihood	Moderate-slide rotational or translation slide due to prolonged wet weather. Potentially greater depth and extent of landslide movement, potentially affecting the proposed building and site users	Rare	Minor	**Low**
Maxim credible	Extensive deep-seat landslide movement due to major seismic event or extreme rainfall event	Barely credible	Medium	**Low**

3.8.1.4 Stage 4 – Application of Risk Assessment Outcomes

The results of the Stage 3 assessment indicate that the overall landslide risk for the site is low.

Accordingly, the activity status for the proposed development is consistent with that of a **low landslide hazard risk** site under Item A114 of Table 36.4.1B of the Auckland Unitary Plan[16].

3.8.2 Building platform retention

The proposed development requires up to 3.5 m fill to be placed at the eastern edge of the site to form the finished building level for Building 1 (RL 16.5 m). This fill is located on sloping ground of approximately 15 to 20°.

To provide adequate ground retention in this location and mitigate excessive surcharging of the slope, the following options may be considered during detailed design:

1. **Reinforced Concrete (RC) or Timber Pile Retaining Wall –** A vertical or near-vertical wall supported by bored or driven piles to retain the fill and resist lateral earth pressures. Suitable where space is limited and a high structural capacity is required.
2. **Gravity Retaining Wall –** A mass concrete, reinforced concrete, or modular block wall that relies on its self-weight to resist earth pressures. Suitable for locations with sufficient footprint for the base width.
3. **Polystyrene with Suspended Floor –** Use of lightweight expanded polystyrene (EPS) fill to reduce loading on the slope, combined with a suspended floor slab to bridge over the lightweight fill. Suitable for reducing settlement and lateral pressures.

Each of the above solutions, subject to detailed design, will effectively mitigate the identified slope stability risks. The preferred option will be confirmed during detailed design once the final site levels, building loading conditions, and spatial constraints are established, allowing the most practical and cost-effective solution to be adopted.

4 Proposed conditions

The proposed conditions of consent are presented in Appendix F in accordance with the Fast-track Approvals Act 2024 for the 'The Point Mission Bay' project. These conditions of consent have been recommended to manage the effects of the proposal, and are consistent with the Council's standard conditions.

[16] https://new.aucklandcouncil.govt.nz/content/dam/ac/docs/plans/unitary/pc-120/52-pc120-chapter-e-natural-hazards-and-flooding.pdf

5 Conclusions

Based on the geotechnical investigations undertaken, the following preliminary geotechnical conclusions have been made in relation to the project:

1. The application site is underlain by approximately 0.1 m to 2.8 m of fill, overlying East Coast Bays Formation (ECBF) residual soils and weathered ECBF. ECBF rock was encountered at depths ranging from 3.0 m to 10.5 m.
2. The site subsoil category for seismic design should be taken as **Category C** (shallow soil site).
3. A shallow raft bearing on residual ECBF soils/weathered ECBF can be considered for the building foundations, subject to detailed design being undertaken to confirm that differential settlements will remain within acceptable limits. Ground improvement or settlement reduction piles can be considered to manage settlements if required.
4. For concentrated building loads or for uplift loads, bored reinforced concrete piles founded in the underlying ECBF rock can be utilised. Further investigations during detailed design are required to confirm rock depth across the site prior to piling.
5. The perched groundwater table is located at approximately 2.0 m to 4.5 m depth. The proposed basement excavation will extend to depths of 1.5 m to 5.0 m below this perched groundwater level.
6. A groundwater drawdown assessment has been undertaken based on groundwater monitoring results and the proposed development plan, which includes permanently drained basements. The assessment indicates that up to 5 mm of settlement may occur during basement excavation due to drawdown, which is below generally accepted limits for buildings and services.
7. Temporary retaining walls will be required for excavations near site boundaries, particularly along the northern boundary. Elsewhere, temporary battered slopes or retaining walls may be viable depending on local geometry and construction staging, subject to detailed design.
8. The temporary basement retaining wall system, consisting of reinforced concrete (RC) piles, both cantilevered and proposed, has been analysed using the WALLAP software. The assessment results are presented in Table 3.14.
9. Ground movements due to excavations and groundwater drawdown have been calculated. Effects upon nearby buildings are expected to be negligible.
10. A preliminary subgrade CBR of 3% may be adopted for design of floor slab on grade and pavements.
11. Significant volumes of earthworks will be required to achieve the proposed finished levels. Conditions related to cut and fill earthworks are set out in Appendix F.

6 Further work

Developed / detailed design:

- Detailed shallow foundation analyses for raft foundations (where applicable) to iterate foundation springs for structural design of the foundation.
- Lateral soil springs for various pile diameters and ground beams / walls for use by the structural engineer for lateral response of the structures.
- Vertical pile springs for use by the structural engineer for assessing seismic response of the structures.
- Temporary work design to confirm the proposed temporary retention scheme for the basement excavation.
- Earthworks specification should be prepared prior to construction

Construction:

- Proof drilled boreholes may be undertaken to better quantify the level of the ECBF rock (where piles are adopted) to de-risk pile cage length uncertainty.
- Construction inspections including inspections of piles / foundations, earthworks and subgrade for confirmation of geotechnical assumptions and subsequent issuing of PS4.

7 Applicability

This report has been prepared for the exclusive use of our client Ngāti Whātua Ōrākei Whai Rawa Limited and Generus Living Group Ltd, with respect to the particular brief given to us in respect of the resource consent application for submission to the EPA under the Fast-track Approvals Act, and it may not be relied upon for any other purpose.

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

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

8 Compliance with the Environment Court Practice Note 2023

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

I am employed by Tonkin & Taylor Limited. I hold the BE (1989) and ME (1993) from the University of Auckland. I am a Chartered Member of Engineering NZ, a Fellow of Engineering NZ. I have 36 years experience (post bachelor degree) in civil and geotechnical engineering

Recent relevant projects and services that I have been involved with include The Foundation Development (in Parnell), The Horizon Development (Mission Bay) and Devore St Apartment (St Heliers).

Authorised for Tonkin & Taylor by

[redacted]

Andrew Langbein
Project and Technical Director - Geotechnical

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