Kāinga Ora Avondale Precinct

Preliminary Geotechnical Desktop Report

Beca Ltd

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Contents

Appendices

Figures

Tables

1 Introduction

Kāinga Ora, formally Housing New Zealand Corporation have 38 sites proposed to be part of redevelopment to provide new state housing in Avondale, Auckland (Figure 1). The development has the potential to provide an estimated 4,413 dwellings on nearly 50ha of land. The dwellings will typically be two to three storey NZS3604 structures but are proposed to extend up to eight storeys towards Great North Road to the east.

Beca Ltd has engaged Aurecon New Zealand Ltd (Aurecon) to provide geotechnical engineering services as part of a high-level risk assessment Kāinga Ora Avondale property to assist future redevelopment. This desktop report provides preliminary geotechnical desktop assessment of the site and recommendations for the proposed redevelopment. Preliminary contamination assessment and commentary has been provided in a separate environmental report.

A suitably qualified and experienced geotechnical engineer should be consulted in both the detailed design and construction stages. The Geotechnical engineer should verify soil strength at the founding level, undertake pile inspections and assess slope stability issues. This report is intended to assist in master planning and preliminary cost estimating and should not be used for consenting or design.

Figure 1 Proposed development boundaries, Avondale, Auckland

Note: Aerial imagery sourced from LINZ Data Service, Auckland 0.075m Urban Aerial Photos (2017) and licenced under the Creative Commons Attribution 4.0 New Zealand Licence. URL: https://data.linz.govt.nz

2 Scope of Works

The objective of this desktop review was to carry out a preliminary desktop assessment of ground and groundwater conditions at the site. Our scope comprised the following:

- Review of published geological and geotechnical information, and relevant investigation data held in Aurecon archives, the New Zealand Geotechnical Database, and information supplied by Kāinga Ora.
- High-level review of the proposed development and preliminary assessment of geotechnical risks.
- Preparation of this preliminary geotechnical desktop report with comments and recommendations for the proposed redevelopment.

3 Kāinga Ora Sites Summary

3.1 Precinct Summary

Table 1 outlines the key details of each of the 38 Kainga Ora sites in the Avondale Precinct and which geological zone we have applied to each development. See Figures 2 and 3 for the location of each site and the assumed geological zones for high-level assessment of the geology for the purposes of the report. The Avondale Precinct plans are presented in Appendix A.

Table 1 Summary of Kāinga Ora Avondale sites

ID	Address	Area (m^2)	Proposed Storeys	Estimated Yield[1]	Geological Zone
1	s 9(2)(a) [redacted]	26,772	3	241	1
2		1,354	3	8	1
3		675	2	3	1
4		12,488	5	156	1
5		609	2	2	1
6		5,018	3	45	1
7		683	5	9	1
8		2,041	5	26	1
9		607	2	2	2
10		1,408	3	13	2
11		1,439	3	13	2
12		1,172	3	7	2
13		1,012	3	6	1
14		2,145	3	19	1
15		1,884	3	17	1
16		4,615	3	42	1
17		1,262	3	11	1
18		710	2	4	1
19		675	2	3	1
20		7,468	5	93	1
21		675	3	6	1
22		5,665	6	85	4
23		6,625	5	83	4
24		3,072	6	46	4
25		498	2	3	2
26		652	2	3	2
27		7,474	3	67	2
28		4,401	3	40	2
29		917	5	11	3
30		9,647	8	180	3

ID	Address	Area (m²)	Proposed Storeys	Estimated Yield[1]	Geological Zone
31	s 9(2)(a) [redacted]	8,464	8	158	3
32	[redacted]	8,615	8	161	4
33	[redacted]	1,212	6	18	3
34	[redacted]	2,912	6	44	4
35	[redacted]	885	5	11	3
36	[redacted]	1,507	6	23	4
37	[redacted]	4,433	8	83	4
38	[redacted]	357,600	8	2,532	3

[1] Yield based on estimates using National Policy Statement (NPS) density uplift method

s 9(2)(a)
[redacted]

s 9(2)(a)

[redacted]

3.2 Precinct Description

The proposed Kāinga Ora Avondale Precinct is located in the suburb of Avondale, Auckland and is west of the city centre. The suburb is largely made up of residential zoning, with little commercial zoning present. The precinct comprised a total of 38 sites which are expected to provide approximately 4,413 dwellings for the use of the state over nearly 50 hectares of land.

The biggest of all these sites is Site 38 which a total area of approximately 35 hectares and has historically been used as a racecourse before being closed in recent years. The site is mostly grassed except for the Avondale Jockey Club buildings to the north of the track. This site is reasonably flat, typically slope in the order of 1:100 (based on Auckland City Council GIS maps) with much steeper slopes on the south and western boundaries.

To the west of the racecourse, there are four sites located on Ash Street. These sites are situated near the Whau River and are more steeply sloped than the racecourse. The bank adjacent to the river is estimated to be approximately 45 degrees based on online GIS maps. Almost all of these sites have at least one existing dwelling which would need to be removed prior to any development.

To the north and northwest of the racecourse are Sites 1 through 21. These sites vary in size between 607m^2 and 26,772m^2 are concentrated around Ash Street, Canal Road and Riversdale Road. The properties are already developed, and existing dwellings would need removing prior to any future developments. On a regional scale, these areas slope gradually towards the Whau River.

To the east and northeast of the racecourse are the remaining 12 sites which vary in size between 885m^2 and 9,647m^2. Only one of these sites (Site 37) is on the eastern side of the Avondale Rail Station on the West Rail Line. The properties are also already developed and existing dwellings would need removing prior to any future developments. The topography in the area of the properties slopes more steeply towards the Whau river than elsewhere within the Avondale Precinct development. The Mt Albert Volcano lies to the east of the precinct.

4 Historic Information

4.1 Aerial Photography

The Avondale area has a long history of building development. We have collated historical information relating to the site from different sources and summarised by the images presented in Appendix B and the information in Table 2 below.

Table 2 Summary of historic aerial photography

Figure	Date	Description
A1	22/04/1940	Aerial Photograph of the Avondale area. The racecourse has been constructed as per modern layout. Residential development is concentrated around Great North Road, with some small agricultural land use present to the north of the racecourse.
A2	09/09/1955	Aerial Photograph of the Avondale area. Minor land use shift towards residential. Avondale College has now been constructed.
A3	14/04/1972	Aerial Photograph of the Avondale area. Almost all the agricultural land has been repurposed to residential zoning. Residential roads have been extended west to near the Whau River.
A4	01/02/1988	Aerial Photograph of the Avondale area. All the agricultural land near the site has been repurposed to residential zoning.

4.2 Historic Investigation Data

We have reviewed the available information from geotechnical investigations carried out within close proximity to the proposed Avondale Precinct development. A summary of the reviewed information is presented in in Table 3 below.

Table 3 Summary of historic investigations

Geological Zone	Report or Source	Summary of Report
1	New Zealand Geotechnical Database (NZGD)	Four boreholes to a maximum depth of 7.5m and 11 hand augers to a maximum depth of 5m. Data has poor coverage across this zone as it is concentrated near Site 1. The logs indicate that very soft silts, clays and peat were encountered with isolated pockets of sand.
2	NZGD	One borehole to 9.4m depth and six hand augers. Data has poor coverage across this zone as it is concentrated near Site 1. The logs indicate that very soft silts, clays and peat (Puketoka Formation) overlying weathered insitu rock at relatively shallow depth. Very low investigation coverage in this area.
3	NZGD, Tonkin & Taylor (1998), Geoconsult (2017)	These reports outline the general subsurface geology comprises topsoil/fill overlying clays, silts and peat (Puketoka formation) and insitu rock (East Coast Bays formation). Investigation data is very sporadic, and no information is present within the racecourse ring.
4	NZGD	Several boreholes to a maximum depth of 100m were undertaken along the eastern extent of the precinct development. The logs indicate a shallow fill layer overlying residual East Coast Bays Formation. Some logs suggest a layer of Puketoka formation at the surface.

5 Site Conditions

5.1 Regional Geology

The regional geology for the area is described in the 1:50,000 (Kermode, 1992) and 1:250,000 (Edbrooke, 2001) scale geological maps of the Auckland Area and the associated publication (refer to Figure 4 and 5 respectively). The Avondale Precinct is located about 1km west of the Auckland Volcanic field.

Geological Zone 1 to 3

The 1:50,000 geological map indicates that the geomorphology of site is the *Puketoka Formation (tp) - light grey to orange brown pumiceous mud, sand, and gravel with black muddy peat and lignite.* The 1:250,000 geological map indicates that the geomorphology of site is as per the 1:50,000 map but uses different notation *(Pup).*

Geological Zone 4

The 1:50,000 geological map indicates that the geomorphology of site is the *East Coast Bays Formation (re) - greenish grey, alternating muddy sandstone and mudstone, with occasional interbedded lenses of grit.* The 1:250,000 geological map indicates that the geomorphology of site is as per the 1:50,000 map but uses different notation *(Mwe).*

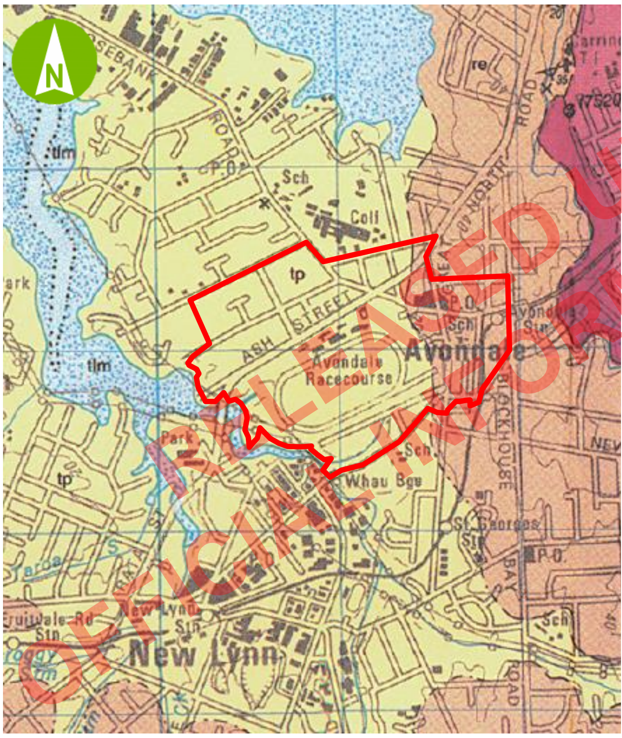


Figure 4 1:50,000 Auckland geology map (Kermode, 1992)

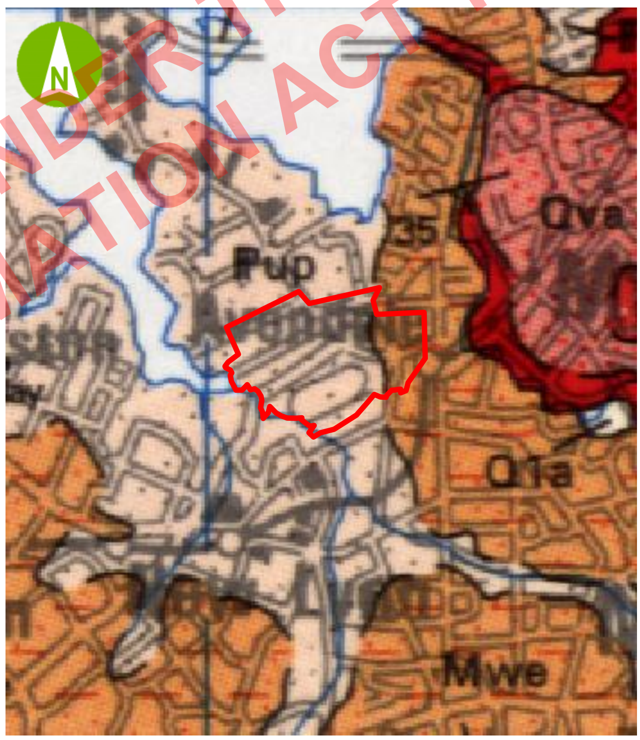


Figure 5 1:250,000 Auckland geology map (Edbrooke, 2001)

5.2 Active Faults

The GNS database indicates that the closest active faults from the site are as follows:

- Wairoa North Fault approximately 30km southeast of the site
- Wairoa South Fault approximately 46km southeast of the site
- Kerepehi Fault approximately 82km southeast of the site

None of these are Major Faults as per NZS 1170.5. The Wairoa North and South Faults have an unknown fault sense, unknown recurrence interval, last event, slip rate and single event displacement, While the

Kerepehi fault has an unknown fault sense, Level 2 recurrence interval (>2,000 to <=3,500 years), last event (millennium), slip rate (low) and unknown single event displacement (GNS, 2020).

5.3 Other Hazards

The Auckland Emergency Management and Auckland Council hazard map for the area shows the following hazards for the area (source: **https://www.arcgis.com/apps/MapSeries/index.html?appid=81aa3de13b114be9b529018ee3c649c8** - Viewed 10 November 2020):

- Volcanic Zone: Within 5km buffer zone of the current Auckland Volcanic Field
- Flood zone: Several sites across all zones are within flood plains and/or flood prone areas. The majority of these areas are concentrated adjacent to Whau River/Avondale Stream, racecourse and Riversdale Road.
- Wind zone: Low wind zone
- Tsunami evacuation zone: Sites 25, 27 and 28 (Zone 2) are within tsunami evacuation zone yellow (covers the largest area that would need to be evacuated in the event of a maximum-impact tsunami)

It should be noted that the above geological, fault and hazard maps are regional in nature and the information indicated on them does not necessarily specifically apply to the site.

5.4 Subsurface Conditions

Across the site extent there are few geotechnical investigations that can be used to predict and constrain the subsurface geology. Given the large extent and the lack of information, the site has been split into four geological zones to assist in characterising groups of sites at a high-level. Local variations within units and lateral extent will occur, therefore the following profiles are general in nature to create a better understanding of the site and to assist with recommendations for the redevelopment of the Avondale Precinct. The main geological units across the site are:

- Topsoil and/or fill – Highly variable in extent and nature
- Puketoka Formation (Tauranga Group alluvium) – Typically alluvial soils comprising moderate to high plasticity, silty clay and clayey silt interbedded with low organic silt and peat with wood fragments. The soil is typically moist, soft to firm with traces of fine sand.
- East Coast Bays Formation (Waitemata Group) – Weak to very weak interbedded sandstone/siltstone with variable weathering. The upper surface typically comprises a surficial layer of residual soil and becomes less weathered (and more competent) with depth.

5.4.1 Geology in Zone 1

The historic borehole logs show that a shallow layer of topsoil directly overlies the Puketoka Formation. The Puketoka formation contains moderate to high plasticity silts, clays and peat with occasional sand layers. The logs show that the soils are very soft / very loose to at least 7.5m. The soils appear to have low bearing capability which will likely necessitate specific engineered design for the foundations. The depth to the East Coast Bays formation is unknown but is unlikely to have any impact on the development in the zone.

5.4.2 Geology in Zone 2

The historic investigation logs show that a shallow layer of topsoil/fill directly overlies the Puketoka Formation. The Puketoka formation contains silts and clays that appear to be of lower plasticity than in Zone 1. The logs indicate that the Puketoka formation soils are very soft to firm. The soils appear to have low bearing capability which will likely necessitate specific engineered design for the foundations. The depth to the East Coast Bays formation varies in the order of 6.3m to 16.3m based on historic borehole logs. This zone is considered unlikely to meet the requirements of 'good ground' as in NZS 3604.

5.4.3 Geology in Zone 3

The historic investigation logs indicate that the site is likely to comprise a thin layer of topsoil/fill underlain by the Puketoka formation. The Puketoka formation contains moderate to high plasticity silts, clays and peat with occasional sand layers. The logs show that the soils are very soft to stiff in the upper 5m. The soils appear to have low bearing capability and which will likely necessitate specific engineered design for the foundations. The depth to the East Coast Bays formation is unknown but is likely to be at least 8m depth and increasing in depth to the south.

5.4.4 Geology in Zone 4

The historic investigation logs indicate that the site is likely to be underlain by a thin layer of topsoil/fill underlain by the Puketoka formation. The Puketoka formation contains moderate to high plasticity silts, clays and peat with occasional sand layers. The logs show that the soils are very soft to stiff in the upper 5m. The soils appear to have low bearing capability which will likely necessitate specific engineered design for the foundations. The depth to the East Coast Bays formation is unknown but is likely to be at least 8m depth and increasing in depth to the south.

5.5 Groundwater

The historic investigation logs show that the groundwater level varies between 0.5m and 3m below existing ground level across the Avondale Precinct. In some areas, the groundwater will need taken into account to mitigate its effect during excavation or foundation construction. The prevailing groundwater gradient slopes down from the northeast to the Whau River and the Waitemata Harbour.

It should be noted that groundwater is variable both spatially and temporally and the depths recorded are likely to differ following periods of heavy or prolonged rainfall or drought.

6 Engineering Considerations

6.1 General

The proposed Avondale Precinct development (see Figure 2) is to provide large scale state residential housing and potentially other community amenities (i.e. primary school). The residential housing is likely to be either:

- Terraced Houses – Two to three storeys, with a shared wall and typically either lightweight timber construction or tilt-up concrete slabs or blocks. Likely to be NZS 3604 structures
- Apartments – Generally greater than three storeys, with self-contained dwellings accessed via common circulation areas. These will require specific engineered foundations

The preliminary geotechnical considerations for future planning, programming and the safe construction of the Avondale Precinct development include:

- Foundation types and recommendations
- Slope stability
- Expansive soils
- Site subsoil classification in accordance with NZS1170.5:2004– Structural Design Actions Part 5 - Earthquake (including amendments)
- Liquefaction potential

Each of these considerations are discussed below. It should be noted that the comments and recommendations are based on regional geological information and historical investigations and should be treated as preliminary in nature. We recommend that site-specific geotechnical investigations be carried out to further refine geotechnical risks in the precinct and assist with specific engineered design or suitability with NZS 3604 where appropriate.

6.2 Foundation Options

Considering the scope of proposed development works, shallow foundations are unlikely be feasible because of weaker soils near the ground surface, especially for the larger apartment blocks. Some of the possible foundation options are below subject to site-specific investigations and advice:

- Suspended Floor – Lightweight timber structures compliant with NZS3604:2011 (Timber-Framed Buildings not requiring specific engineering design) could be supported on simple timber post footings embedded in concrete with suspended floors.
- Rib Raft – Where soft and/or compressible alluvial soils are encountered; a rib raft system could be used for lightweight timber structures (detached or low-rise terraced houses). The rib raft system acts to reduce differential settlements, both static and seismically induced.
- Raft Foundations – For heavily loaded structures, raft foundations with ground improvement could be adopted as an alternative to pile foundations if depths to competent soils/rock is greater than 15m. Raft foundations are effective in reducing the differential settlement and resisting any uplift loads. Coupled with ground improvement, this could be an economical option.
- Pile Foundations – For heavily loaded structures (apartments) piles to competent material, either into more competent Puketoka or East Coast Bays formation may be required to resist design loads and reduce settlements. The presence of very weak soil layers such as peat and organic clays may cause negative skin friction effects in piles. The relevant piling options are discussed in Table 5 below.

Table 4 Pile option summary

Pile Type	Principle	Pros	Cons	Comments
Concrete bored piles founded on competent soils at depth	Pile auger hole excavated to required depth, steel cage dropped in the hole and concrete poured via Tremie method. The holes may need to be stabilised during construction using either temporary/permanent casing or bentonite/polymer	Common and robust foundation solution Relatively straightforward quality control and quality assurance method Relatively high lateral capacity Flexibility in terms of depth and diameter	Will generate spoil. Proof tests need to be carried out at pile locations to confirm the founding layer. Presence of relatively large gravels or cobbles can hinder pile construction although not likely to be a problem at the site	Potential solution
Continuous Flight Auger (CFA) piles	Hollow stem pile auger hole excavated to depth, concrete tremie pour as auger stem is removed. Steel cage is plunged once auger head is clear of hole.	Significantly quicker construction methodology Temporary or permanent hole stabilisation is likely not required Low noise/vibration construction methodology	Limited diameter and depth options and capacity. Will generate spoil Proof tests need to be carried out at pile locations to confirm the founding layer Poor quality assurance of founding layer suitability	Potential but not preferred solution
Screw piles	Hollow steel sections with helix on end are screwed into the ground.	Quiet and quick installation without any spoil. Straightforward quality control and quality assurance method (each pile tested indirectly via the installation torque giving a reliance on the uplift capacity).	Unable to penetrate into rock, also presence of large gravels and cobbles will hinder the installation with the risk of helix bending or not achieving design depth Very low lateral capacity Proprietary design and construction with limited number of firms doing such work. Installation and design dependent on skill of contractor	Potential solution
Driven piles	Driving steel UC section	Quick and simple installation Capacity can be verified with Hiley formula or PDA testing	Head height, noise and vibration issues. Small size (end bearing and skin friction) limited compared with other pile options. Reliability of skin friction capacity under cyclic earthquake loads High risk of not achieving adequate embedment into rock due to penetration refusal	Potential but not preferred solution

6.3 Slope Stability

The only notable slopes across the precinct are on the edges of the racecourse and adjacent to the Whau River. The slopes are likely to be formed of Puketoka and East Coast Bays formations. Based on the existing information, the slopes appear to be stable, however, the long-term stability of the slopes should be assessed once the ground and groundwater conditions on the slope are better constrained and the likely surcharge any building developments is known. The stability of these slopes is likely to be controlled by shallow sliding of overlying alluvial or residual soils.

We recommend, that houses should not be founded immediately adjacent to the more steeply sloped areas, without careful consideration of slope stability and appropriate setback distances from the slope. Further investigation and engineering analysis would be required to clarify these limitations in conjunction with earthworks and retaining assessments.

Where slopes are to be left undeveloped and/or vegetated, we recommend a maximum preliminary slope angle of 1V:3H be adopted. To maximise the yield of the development, retention solutions could be considered on the western and southern edges of the racecourse site.

6.4 Expansive Soils

Expansive soils swell and contract with changes in moisture content, typically due to seasonal variability in groundwater, which can result in ground deformations and increased loading on structural foundations. The Puketoka formation found almost everywhere within the proposed precinct contains layers of peat and organic clay. Presence of highly plastic silty clay and peat can exhibit expansive soil behaviour. However, definite conclusions can only be drawn once the site-specific investigation is completed.

Portions of the Auckland Region is known to be underlain by soils classified as “expansive” in accordance with AS2870: 2011 Residential Slabs and Footings (BRANZ, 2008). Presence of such soils near surface precludes standard NZS3604 type foundations and specific engineering design would be required.

6.5 Site Subsoil Classification

The New Zealand Loading Standards NZS1170.5:2004 categorises five site classes (Class A to Class E), Class A being classed as “strong rock”, and Class E as “very soft soil”. We have assessed the site flexibility based on the following:

- Historic investigation data summarised in Table 3 of this report
- Clause 3.1.3 and Table 3.2 of Structural and Design Actions: Earthquake Actions NZS 1170.5: 2004

Zone 1

We consider the preliminary site subsoil classification in terms of NZS1170.5:2004 Clause 3.1.3 to be Class C (shallow soil sites) in general. Should the depths in Table 3.2 of this standard be exceeded than Class D (deep or soft soil sites) should be used. In some of the historic boreholes, up to 7m of very soft soils were encountered (boreholes did not advance beyond this depth) therefore it may be possible that some sites could be characterised as Class E (very soft soil sites).

Zone 2 and 3

We consider the preliminary site subsoil classification in terms of NZS1170.5:2004 Clause 3.1.3 to be Class C (shallow soil site). Should the depths in Table 3.2 of this standard be exceeded than Class D (deep or soft soil sites) should be used.

Zone 4

We consider the preliminary site subsoil classification in terms of NZS1170.5:2004 Clause 3.1.3 to be Class C (shallow soil site) in general. Should the competent rock be encountered near the ground surface (less than 3m as per the conditions in NZS1170.5) then Class B (rock site) could be used.

6.6 Liquefaction Potential

Under cyclic shaking loose and non-plastic soils such as sand and coarse silt tend to decrease in volume due to densification. If these soils are saturated and rapid shaking occurs under un-drained conditions, the soil densification causes pore water pressure to increase. The increase in pore water pressure results in a loss of soil strength due to a decrease in effective stress, and eventually leads to liquefaction once effective stress drops to near zero. Liquefaction can lead to large displacements and bearing capacity failure of foundations, slope failures and sand boils.

The three primary factors that contribute to liquefaction are:

- Saturation of soils i.e. high groundwater table
- Loose and uniformly graded soils e.g. sand, coarse non-plastic silt, etc.
- Strong earthquake shaking

Each of these is considered below together with initial conclusions on the site liquefaction potential.

Saturation

Fully saturated soils are susceptible to liquefaction. The groundwater levels were measured between 0.5m and 3.0m below existing ground level across the precinct. Hence soils below these depths may be susceptible to liquefaction.

Soil Composition

Liquefaction typically occurs in loose and uniformly graded sands and low plasticity silts with low to moderate permeability. Loose gravels can liquefy if they have a low permeability and are confined by less permeable soils.

The historic borehole logs typically describe SILT, clayey SILT and silty CLAY soils with interbedded organics and typically high plasticity. Some investigation logs describe discrete beds of moist, non-plastic fine to medium sand less than 0.5m thick which have the potential to liquefy.

Earthquake Intensity and Ground Shaking

Auckland is a region of relatively low seismicity with limited potential for ground shaking. The NZGS (2016) Guideline gives a procedure to calculate design earthquake magnitudes and peak ground accelerations for New Zealand sites. Using this guideline, we have assessed the following Serviceability Limit State (SLS) and Ultimate Limit State (ULS) design events for a range of subsoil classification in Table 4.

Table 5 Design PGA and earthquake magnitudes

Design Life	Building Importance Level	Earthquake Return Period	Site Subsoil Classification	a_{max}	Magnitude
50 years	Level 2 (assumed)	1 in 500 years (ULS)	B	0.12g	5.75
			C	0.155g	
			D and E	0.155g	
		1 in 25 years (SLS)	B	0.03g	5.75
			C	0.04g	
			D and E	0.04g	

Upon review of the existing logs, the Puketoka formation comprises predominantly of silts and clays with high plasticity which implies very low liquefaction risk. Even the interbedded fine sand layers, which are more likely prone to liquefaction are thin. Based on this preliminary assessment, we can conclude the liquefaction risk is low for this site.

Due to the low density of geotechnical information and inherent variability of the geology in the area, the liquefaction potential across each site should be assessed individually once the site-specific investigation is undertaken.

7 Recommendations

Our recommendations for the Avondale Precinct development are as follows:

- Over a majority of the 38 sites there is very little to no geotechnical information available. To better provide geotechnical assessment and confirm suitable foundation philosophy and design, site-specific investigations are required. These investigations should comprise boreholes, Cone Penetration Tests (CPT) and hand augers with Scala probes and shear vane tests. Soil samples should be collected for geotechnical laboratory testing. We note the final scope of investigation should be determined after preliminary planning at each site has occurred so the tests can be targeted to a specific design.
- A detailed, site-specific liquefaction assessment should be undertaken upon completion of a site-specific investigation.
- Changes in moisture content due to seasonal variations could result in shrinkage/swelling in alluvial soils (Puketoka formation) which could result in ground settlements or heave. Laboratory testing should include Atterberg Limits, Linear Shrinkage and Shrink/Swell testing to assess the soil expansivity.
- Any proposed temporary or permanent retaining structures should be designed in consultation with the geotechnical engineer and lateral movement of the wall should be limited as not to cause excessive deformations or cracks in the adjacent buildings/pavements.
- Subgrade strength should be assessed for driveways and parking areas to ensure they are suitable for vehicular loading. It is anticipated that the subgrade CBR will be in the order of 1% to 2%, depending on founding conditions and depth. It is likely that flexible pavement will be the best option due to better performance with settlement in the soft subgrade.
- A suitably qualified and experience geotechnical engineer should be consulted in both the detailed design and construction stages. The Geotechnical engineer should verify soil strength at the founding level, undertake pile inspections and assess slope stability issues, etc.

8 Safe in Design (SiD) Considerations

Specific geotechnical considerations for Safe Design include (but not limited to) the following:

- **Excavations –** Should the proposed development require excavations. Care should be taken to ensure the stability of the excavations and protection of personnel against falling from one level to another (i.e. temporary barriers, signage).
- **Stable Working Platform –** The construction of larger apartments (i.e. 6 to 8 storey) may require piling plant to be mobilised to site. Care should be taken to ensure adequate bearing of the existing asphalted road surface and the construction of additional reinforced working platforms, if required.
- **Bored Pile Installation** – The construction methodology for bored piles should be developed considering groundwater, pile hole stability and movement of piling plant around the site. We recommend the pile holes be protected with either temporary barriers or extended casings a minimum 1.0m above the working platform. Proof drilling of the pile holes, prior to construction, will confirm the adequacy of the founding rock above and below the pile toe, including presence of any voids within the pile length and below the pile toe that may impact the foundation design.
- **Screw Pile Installation –** Should a screw pile foundation be adopted, the size and length of screw piles will need to be considered with respect to transportation to site, movement around site, plant required to install the screw piles and equipment required to carry out load testing.
- **Construction Induced Liquefaction or soil softening –** Liquefaction or soil softening can be triggered by non-earthquake related events, where shear stress in the soils is able to develop. This can occur through excessive vibrations through piling, vibro-installation of pile casings or movement of heavy vehicles.
- **Asbestos Survey –** An asbestos survey should be undertaken in the existing units before demolition.
- **Adjacent Foundations –** All excavations adjacent to existing foundations should be assessed with respect to their foundation stability.

9 Explanatory Statements

We have prepared this report in accordance with the brief as provided. The contents of the report are for the sole use of the Client and no responsibility or liability will be accepted to any third party. Data or opinions contained within the report may not be used in other contexts or for any other purposes without our prior review and agreement.

The recommendations in this report are based on data collected at specific locations and by using suitable investigation techniques. Only a finite amount of information has been collected to meet the specific financial and technical requirements of the Client's brief and this report does not purport to completely describe all the site characteristics and properties. The nature and continuity of the ground between test locations has been inferred using experience and judgement and it must be appreciated that actual conditions could vary from the assumed model.

Subsurface conditions, such as groundwater levels, can change over time. This should be borne in mind, particularly if the report is used after a protracted delay.

This report is not to be reproduced either wholly or in part without our prior written permission.

10 References

Auckland City Council online GIS maps: **https://geomapspublic.aucklandcouncil.govt.nz/viewer/index.html** - Viewed 12 November 2020

Auckland Emergency Management and Auckland Council hazard map: **https://www.arcgis.com/apps/MapSeries/index.html?_appid=81aa3de13b114be9b529018ee3c649c8** - Viewed 11 November 2020):

BRANZ, 2008. Addendum Study Report – Soil Expansivity in the Auckland Region.

Edbrooke, S.W., 2001. Geology of the Auckland area. Scale 1:250,000. Institute of Geological and Nuclear Sciences geological map 3.

Kermode, L.O. 1992: Geology of the Auckland urban area. Scale 1:50,000. Institute of Geological & Nuclear Sciences geological map 2. 1 sheet + 63p. Institute of Geological & Nuclear Sciences Ltd., Lower Hutt, New Zealand

New Zealand active fault database: https://data.gns.cri.nz/af/, viewed 11 November 2020

NZS1170.0:2002. Australia/New Zealand Standard, Structural Design Actions, Part 0: General Principals. Standards New Zealand, Wellington, New Zealand.

NZS1170.5:2002. Australia/New Zealand Standard, Structural Design Actions, Part 5: Earthquake Actions – New Zealand. Standards New Zealand, Wellington, New Zealand.

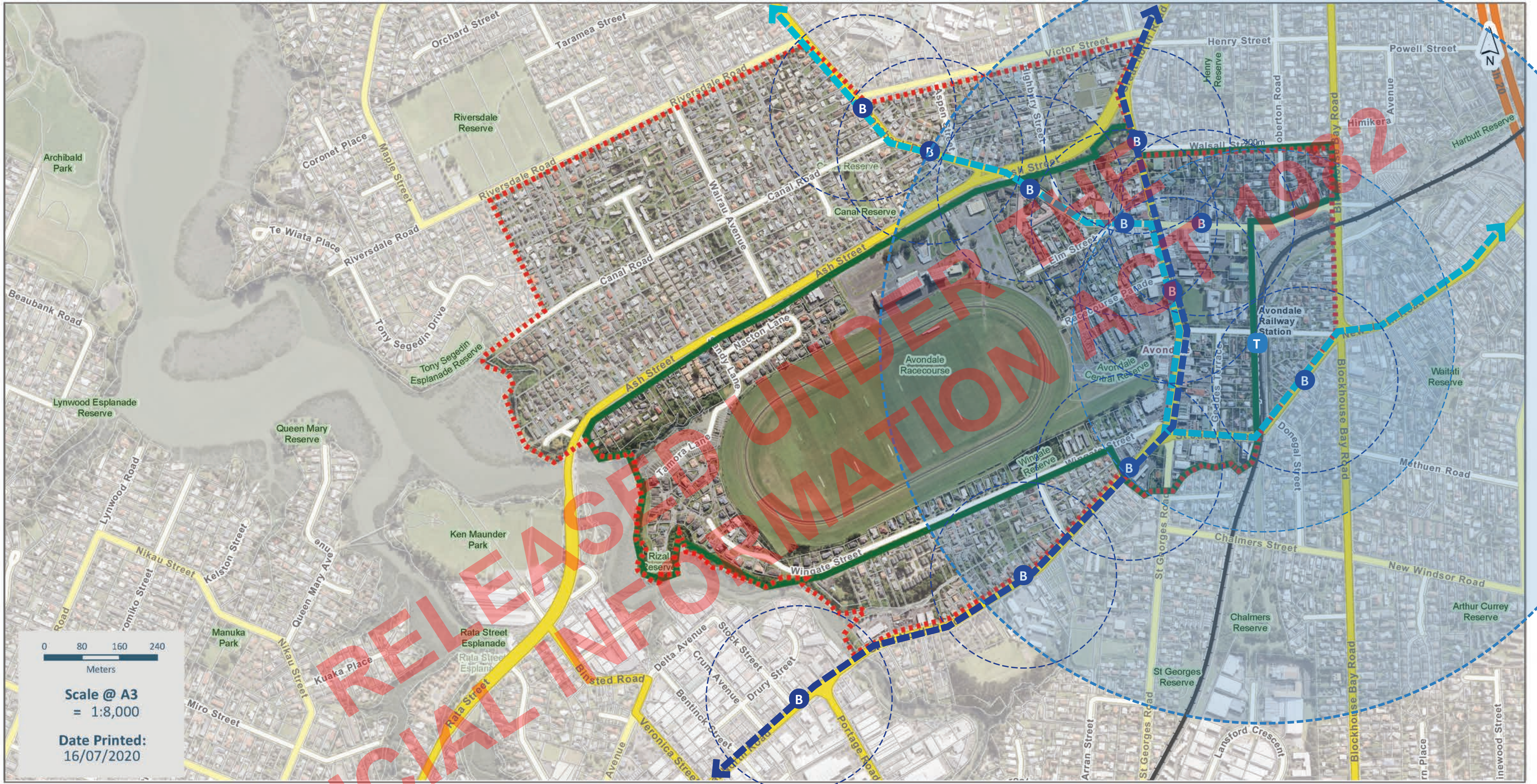
NZGS, 2016. Earthquake Geotechnical Engineering Practice, Module 1 – Overview of the guideline. NZ Geotechnical Society Inc, Wellington, New Zealand.

A

Appendix A

Avondale Precinct Plans

Avondale Strategic Development Study Area



Legend

Influential area boundary

Focused area boundary

Kāinga Ora
Homes and Communities

s 9(2)(a)

[redacted]

s 9(2)(a)

[redacted]

s 9(2)(a)

[redacted]

s 9(2)(a)

[redacted]

B

Appendix B

Historic Aerial Photography

Figure 6 Historic aerial photograph of Avondale, Auckland taken 22 April 1940

Note: Aerial imagery sourced retrolens and licenced under the Creative Commons Attribution 4.0 New Zealand Licence. URL: http://retrolens.nz/

Figure 7 Historic aerial photograph of Avondale, Auckland taken 9 September 1955

Note: Aerial imagery sourced retrolens and licenced under the Creative Commons Attribution 4.0 New Zealand Licence. URL: http://retrolens.nz/

Figure 8 Historic aerial photograph of Avondale, Auckland taken 14 April 1972

Note: Aerial imagery sourced retrolens and licenced under the Creative Commons Attribution 4.0 New Zealand Licence. URL: http://retrolens.nz/

Figure 9 Historic aerial photograph of Avondale, Auckland taken 1 February 1988

Note: Aerial imagery sourced retrolens and licenced under the Creative Commons Attribution 4.0 New Zealand Licence. URL: http://retrolens.nz/

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