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Project Number #25142.000.001

Geotechnical Investigation

Judsons Road and Petries Road, Woodend, Canterbury

Submitted to:
Urban Estates Limited
Level 2, Building 1, 181 High Street
Christchurch Central
Christchurch 8144

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Project No.		25142.000.001	**Doc ID**	04	
Client		Urban Estates Limited	**Client Contact**	Brad Wilson	
Distribution (PDF)		Urban Estates Limited			
Date	**Revision**	**Description**	**Author**	**Reviewer**	**WP**
14/02/2024	0	Draft	JM	RC	DF
27/02/2024	1	Issued to Client	JM	RC	JT

1 Introduction

ENGEO Ltd was requested by Urban Estates Limited to undertake a geotechnical investigation of the property at Judsons Road and Petries Road, Woodend, Canterbury (herein referred to as 'the site'). The purpose of the assessment is to support the plan change application to rezone the site from rural to residential. This work has been carried out in accordance with our signed agreement dated 25 January 2024 (ref. P25142.000.001_01).

Our scope of works was as follows:

- A review of published geotechnical and geological information relevant to the site.
- Site assessment by an experienced ground engineering professional.
- Excavation of 17 test pits to a target depth of 3 m or practical refusal with associated Scala penetrometer and shear vane testing, as appropriate.
- Organisation and technical supervision of 16 Cone Penetrometer Tests (CPTs) to a target depth of 15 m.
- Analysis of field data and production of a conceptual geological site model.
- Production of this geotechnical report based on the findings of our enquiries and ground investigation, including recommendations on the following:
    - Provision of seismic subsoil category based on regional data.
    - High level assessment of geohazards against Section 106 of the RMA.
    - Discussion on water levels and how they may affect the construction of inground infrastructure.
    - Provide geotechnical constraint mapping, if appropriate.
    - Discussion on the viability for residential development and potential foundation / ground improvement options, as appropriate.
    - Recommendations for future geotechnical works.

Our scope of works does not include geotechnical investigations suitable to support Resource, Subdivision or Building Consent.

2 Site Description

The site at Judsons and Petries Road is currently rural farmland with an area of approximately 32 hectares in Woodend, Canterbury. The site is relatively flat with the exception of wooded area at the end of Judsons Road where there is a defined channel that appears to be part of a remnant stream channel. A Site Location Plan is presented in Appendix 1.

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3 Desktop Review

3.1 Regional Geology

The site has been regionally mapped by GNS Science as being underlain by unweathered, variably sorted gravel, sand silt and clay of modern river floodplains or low level degradation terraces. Regional mapping completed by Forsyth et al (2008) indicates that the site is underlain by grey river alluvium beneath plains or low level terraces.

3.2 Geohazards

3.2.1 Seismicity

The nearest active faults to the site are the Loburn and Ashley faults (part of the Ashley Fault Zone), mapped approximately 9 km northwest and 15 km west of the site, respectively. The faults within the Ashley Fault Zone trend roughly east-west, and the fault strands within it are indicated as having equal components of dip-slip and strike-slip movement (Barrell and Van Dissen, 2014). The average recurrence interval of the Ashley Fault Zone is assessed as being between 7,000 and 15,000 years, although it could be as low as 5,000 years. The site is mapped outside of the Ashley Fault Awareness Zone. The Loburn fault has no specific details apart from that it is a dextral slip fault.

Commented [RC2]: Correct?

Commented [RC3]: What about the Loburn fault?

Commented [JM4R3]: Nothing noted on GNS other than it's a dextral fault

Large regional areas of faulting (GNS, 2015) namely the Porters Pass-Amberley Fault Zone, and the Hope and Alpine Faults, are further afield but present a high seismic hazard to the Christchurch area due to the anticipated size of earthquakes generated. The largest of these faults is the Alpine Fault, which has a return period of 250-300 years and is expected to produce a M8 earthquake. The last rupture on the Alpine Fault is believed to have occurred in 1717 (Pettinga et al., 2001).

3.2.2 Canterbury Earthquake Sequence

Ground Shaking

O'Rourke et al (2012) have developed a contour map of the conditional median peak ground accelerations (PGA) interpolated from data measured at various recording stations during the 2010-2011 Canterbury Earthquake Sequence. The nearest monitoring stations to the site are at Kaiapoi North School and Ashley School.

This mapping indicates that the site experienced peak ground motions of approximately 0.21 g during the September 2010 Darfield earthquake. The site is therefore likely to have experienced seismic accelerations in excess of SLS. Contour mapping is not available for the February, June or December 2011 earthquakes.

Liquefaction

The site has been mapped by the Waimakariri District Council as being within an area where 'Liquefaction damage is possible' and further assessment is needed.

3.2.3 Tsunami

Assessing the risk from tsunami is outside of our scope, however, we note the site is located outside of any tsunami excavation zones defined in the Waimakariri District Plan.

3.2.4 Flooding

We have reviewed the Waimakariri District Council GIS database and have presented a snapshot of the map with the approximate site boundary overlain (Figure 1). The mapping indicates that parts of the eastern and southern sides of the site may be subject to a medium flood hazard (defined as inundation depth of greater than 0.3 m) during a 1 in 200 year flood event. A high flood hazard has been associated with the current stream channel for the 1 in 200 year flood event.

Figure 1: Flood Hazard Mapping

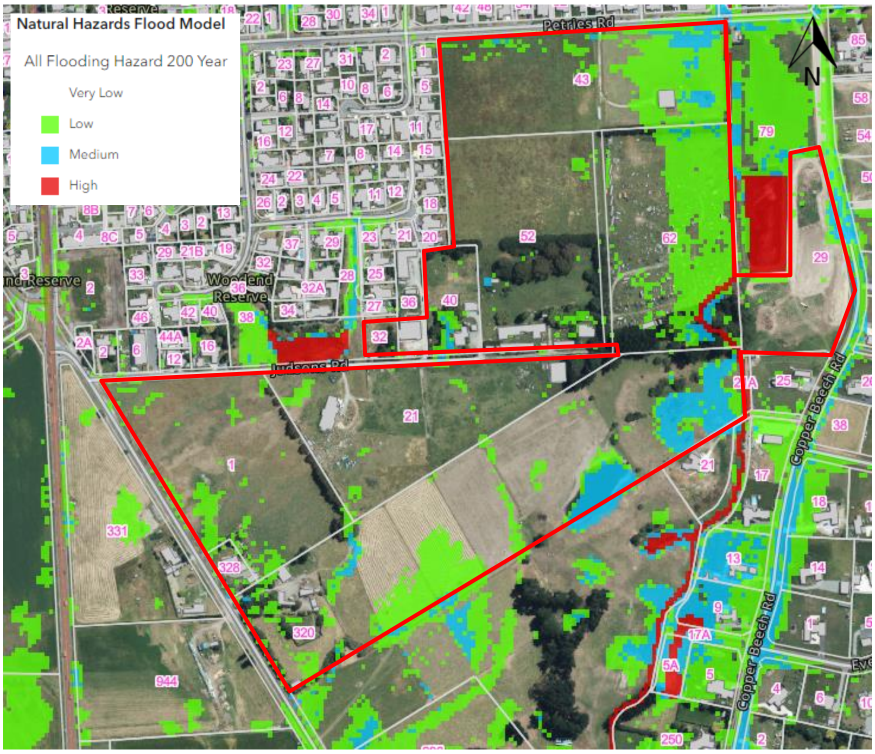

Image sourced from the Waimakariri District council Hazard Maps.

3.3 Historical Aerial Photography Review

We have reviewed historic aerial photographs of the site available through Canterbury Maps (Property Search) dating back to 1940.

The site appears to have been used as agricultural grazing land since the earliest available photograph in 1940. In this photograph a stream channel and associated drainage basin is present in the east of the site, generally running northeast to southwest. The channel enters the site to the northeast of the wooded area at the end of Judsons Road. Paleo channels were also noted on the western side of the site in the 1965-1969 aerial photograph. These are generally orientated north to south. We have highlighted these significant features in Figure 2.

The waterway on the eastern side of the site appears to have been partially infilled between the 1965-1969 and the 1970-1974 aerial photograph. A pond is also noted to the southeast of the site (west of the waterway) in the 1970-1974 aerial photograph. This pond appears to periodically dry out and is still present in the current aerial photograph.

Commented [RC5]: Mark this on your plan below

Commented [RC6]: If this was observed while onsite then we should note that rather than refer to photographs

Commented [JM7R6]: Aerials show that it changed over time

Several structures, inferred to be for agricultural or residential use, were present in all of the historic aerials.

Figure 2: 1965-1969 Historic Aerial

Image sourced from Canterbury Maps.

4 Site Investigation

4.1 Site Observations

ENGEO visited the site on 7 February 2024 to complete a site walkover and made the following observations:

- The majority of the site area was flat with grassed paddock areas used for agricultural grazing. Typical undulations of 0.5 m were noted across the site and generally appear to be natural.
- The site at 29 Copper Beach Road was mounded approximately 2 m higher than Copper Beach Road and we were advised by the owner that topsoil from the road to the east and surrounding subdivision had been placed on this site.
- A small gully was observed in the vegetated area to the south of 62 Judsons Road, no water was present in the gully at the time of the walkover. The banks of the gully ranging from 1 m to 1.5 m in height. The base of the channel ranged from 2 m to 3 m wide.
- A pond area was observed in the eastern section of 320 Woodend Beach Road. The pond was dry at the time of the walkover.

Commented [JM8]: Nat to confirm

Commented [RC9R8]: Let me know once filled in and I will review

Commented [NF10R8]: Done burger

4.2 Test Pits

ENGEO completed technical observation of 17 test pits across the site on 7 & 8 February 2024. The test pits were completed using a 5.5 ton excavator, with test pits reaching depths between 1.8 m and 3 m depth. The test pits that met practical refusal did so on or in a dense gravel layer (TP07, TP14 to TP17).

Commented [JM11]: Check practical refusal on TP07

Full logs are presented in Appendix 2 with their locations in Appendix 1 and are written in accordance with the New Zealand Geotechnical Society field classification guidelines (NZGS, 2005).

4.3 Cone Penetration Testing

The CPT probe gathers raw data including cone tip resistance, friction sleeve resistance, and pore water pressure at 1 cm intervals during the test. This information is used to infer the soil type, soil density and water pressure in undisturbed conditions in the ground and can be used to assess the liquefaction susceptibility of the ground and to calculate geotechnical bearing capacity in the soil.

At our request, McMillan Drilling Ltd pushed 16 CPTs at the approximate locations shown on the test location plan in Appendix 1. Five tests were pushed to a target depth of 15 m, the remaining 11 were pushed until they met practical refusal between 4.83 m and 10.89 m depth. The CPT logs are attached to this report in Appendix 3.

The CPTs on the western side of the site generally met target depth. The remaining CPT's met practical refusal on an inferred shallow gravel or dense gravelly sand layer.

4.4 Groundwater

Seven of the CPT holes were dipped by McMillan Drilling on the completion of each test. The groundwater levels varied between 1.1 m to 4.65 m across the site. CPT08 had the deepest standing water reading of 4.65 m, however this area has been recently filled which may have impacted the water depth. CPT10 encountered the shallowest reading of 1.1 m, this CPT is located within the remnant water way so this was expected, the remainder of the CPT's generally encountered groundwater between 2.3 m and 3.4 m depth.

Groundwater was encountered in the test pits on the western side of the site between 1.6 m and 2.8 m depth. Test pit 17, located within the remnant waterway also encountered groundwater at 2.3 m depth.

ECAN well M35/0546, located approximately 450 m east of site, suggest that groundwater seasonally fluctuates between 2.3 m and 3.7 m.

5 Engineering Geological Model

5.1 Discussion on Inferred Geological Profile

The testing to date indicates that the composition of the subsurface soils is highly variable across the site. This is consistent with the alluvial depositional environment where rivers have avulsed across the landscape over time, creating a layered subsurface profile comprising loose silts, sands and gravel layers. The majority of the site encountered interbedded sands and silts within the upper 4 m of the soil profile. These layers were underlain by dense to very dense sand to approximately 8.4 m depth. It was within this layer that the majority of the CPT's refused.

The western side of the site (CPTs 1,3 & 16) encountered the same profile within the upper 6.5 m of the soil profile. With the exception of CPTs 11 & 12 which encountered an intermediate dense to very dense sand / gravel layer between 1.5 m and 5.5 m depth. Between 6.5 m to 15 m depth the soil profile became interbedded silts and sands. The sands were generally medium dense to very dense, the silts were firm to very stiff.

5.2 Assumptions and Uncertainties

It should be noted that we have had to interpolate the contact between the western test locations, the central test locations and those located within the known remnant waterway on the eastern side of the site. A comprehensive geological model should be formed during the Subdivision Consent stage to assist in informing the master planning decisions for the site.

Due to the limitations of our investigation, the depth to groundwater remains uncertain however in general range across the site has been established dependent on location.

6 Geotechnical Assessment

Based on our review of mapped land damage in the surrounding area of the site and the ground conditions encountered in our investigations, we consider the following geohazards are present on-site:

- Liquefaction induced ground settlement with the possibility of lateral spread towards buried stream channels.
- Presence of potentially compressible near surface soils which may pose a consolidation settlement risk to any proposed development.

These two geohazards are assessed / discussed in the following sections.

6.1 Seismic Subsoil Class

For the purpose of seismic design, we consider the soil classification in line with NZS 1170.5:2004 to most likely be 'Class D – Deep or soft soil sites'.

6.2 Liquefaction Assessment

6.2.1 Soil Liquefaction

Liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The liquefaction potential of a site depends on the presence, and thickness of potentially liquefiable soil (sands and silts below the groundwater table), and the intensity of earthquake shaking at the site.

Liquefaction can lead to settlement of the ground surface, sand boil formation (ejected liquefied material), ground cracking and lateral displacement of the ground surface, slope instability, and differential and vertical settlement of foundations.

We have undertaken a liquefaction assessment using the on-site CPT data and liquefaction procedures described in the following sections.

6.2.2 Ground Motion Parameters for Liquefaction Assessment

We have assessed the likelihood of liquefaction triggering and post-liquefaction induced vertical settlement occurring at the site for design earthquake scenarios in accordance with NZS 1170, Module 1 of MBIE's Geotechnical Engineering Practice Guidance. Based on our understanding of the intended use and capacity of the development we assume that the proposed buildings will be Importance Level 2 (IL2), with a design life of 50 years.

In accordance with NZS 1170.5:2004 we have assessed three limit states as follows and the description of these, including the analysis scenarios are presented in Table 1.

- Ultimate Limit State (ULS)
- Intermediate Limit State (ILS)
- Serviceability Limit State (SLS)

Table 1: Seismic Design Scenarios

Design Case	Seismic Performance Expectations	Assumed Site Class / Importance Level / Design Life	Return Interval	PGA (g)	Magnitude
ULS	Under Ultimate Limit State (ULS) seismic loading the structure should be able to accommodate the potential deformations without structural collapse and protect the safety of the occupants.		500 yrs	0.35	7.5
ILS	Intermediate Limit State (ILS) - The Waimakariri District Plan liquefaction mitigation design standards (Table 32.3) sets a limit of 100 mm of liquefaction induced vertical settlement and 250 mm of lateral spreading at an intermediate limit state of 1 in 150 year event. This earthquake scenario represents an intensity of shaking that is considered to have a high likelihood of occurring within the land use planning horizon.	Class D / IL2 / 50 years	150 yrs	0.20	7.5
SLS	Under Serviceability Limit State (SLS) design seismic loading, the expectation is that deflections do not result in damage causing loss of function of the structure and that damage is readily repairable.		25 yrs	0.13	7.5
				0.19[2]	6[2]

[1]ILS scaled from the ULS design case using the Return Period Factor for IL2 from Table B1 of the MoE Structural and Geotechnical Guidelines.
[2]As per Issue 7, Update 50 of the clarifications and updates to the 2012 MBIE Guidance. This second SLS case should be assessed when using the B&I liquefaction triggering procedure.

6.2.3 Liquefaction Triggering

- Liquefaction triggering method: Boulanger and Idriss (2014) as prescribed by MBIE.
- Design ground motions (detailed in Table 1).
- A threshold probability of liquefaction (P_L) of 16% for design earthquake loading.
- A soil behaviour type index (I_c) cut-off value of 2.6 to differentiate between susceptible and non-susceptible to liquefaction soils.
- For purposes of liquefaction analysis, we have adopted the groundwater level of 2 m for the majority of the CPTs. A groundwater level of 1 m was used for the CPTs located within the remnant waterway. The assumed water level is shown on the analysis outputs.

Consequences of Liquefaction

- Vertical Settlement - The Zhang, Robertson, and Brachman (2002) procedure for estimating volumetric strain and vertical settlement.
- Surface Expression of Liquefaction - We have estimated surface expression of liquefaction (e.g. sand boil formation) through the index parameter Liquefaction Severity Number (LSN).

6.2.4 Liquefaction Analysis Results

Results of our liquefaction analyses are presented in Appendix 4, with the liquefaction severity number (LSN) reports included in Appendix 4b. The calculated settlements for a given limit state are summarised below:

- ULS: 10 mm to 90 mm
- ILS: negligible to 55 mm
- SLS: negligible to 20 mm

Commented [RC12]: Add these in

In terms of site performance (the LSN reports), during the design seismic cases little to no expression of liquefaction is expected under SLS conditions. During an intermediate design case minor expression of liquefaction may be expected within the remnant waterway, with the remainder of the site expected to experience little to no expression of liquefaction. A similar pattern is noted under ULS conditions with the majority of the site expected to experience negligible to minor liquefaction ejecta. The CPTs within the remnant stream channel however could experience moderate liquefaction ejecta.

We consider that the majority of the site falls with the Technical Category (TC) 2 classification (as defined by Canterbury MBIE Module 1), and although some areas exhibit TC1 type performance this is likely due to shallow refusal of the CPTs in these areas. Therefore, we recommend that TC2 type performance is assumed for the whole site.

These analyses could be refined through further testing and ground truthing, and this is discussed in Section 8.

6.3 Lateral Spread

Given the civil / landscaping design of the site wide landform has not yet been developed we have not specifically addressed lateral spread. There are currently unsupported slopes present in the wooded area to the east of Judsons Road that could be susceptible to lateral spreading, including the possibility of lateral movement towards buried streams during an earthquake.

Development of properties near any slopes, either designed or natural, generates a risk for ground surface to move towards the slope during a seismic event when liquefaction occurs. This should be assessed during the Subdivision Consent works.

6.4 Preliminary Consolidation Analysis

We consider the weak cohesive material identified in the CPTs between 0 m and 3.5 m depths may be susceptible to consolidation over time. It is also possible this material is peat which also poses consolidation risk, although no peat was encountered in the test pits.

Consolidation rates can be increased when additional load is placed on the site from either earthworks or new buildings. To estimate the consolidation potential on this site, we have completed a preliminary settlement calculation using the geotechnical software CPet-IT. While we have not completed any consolidation or lab testing of the subsurface materials, we consider this is appropriate as a “first-pass” to indicate the likely consolidation potential of these deposits.

6.4.1 Consolidation Analysis Parameters

As a subdivision earthworks plan has not yet been developed, we have considered a generalised fill load across the approximate area of a residential building as a preliminary assessment of potential consolidation. We have considered the following parameters for the analysis:

- We have assumed that any fill will be placed on the material directly below topsoil. For this preliminary analysis we have considered a 100 m^2 wide fill platform founded directly below topsoil. This has been modelled as a fill pressure of 18 kPa acting at ground level.

- This scenario has been modelled for six months of primary settlement and 600 months (50 years) of secondary settlement, which we assume is the design life of the structures within the proposed development.

6.4.2 Analysis Results

The analysis suggests that the majority of the site has negligible susceptibility to consolidation settlement of the soft cohesive material identified in many of the CPTs, with less than 25 mm predicted over a 50 year design life for the majority of the site. Only two CPTs (11 & 12) indicated settlement values in excess of 25 mm and these were marginal (29 and 25 mm, respectively).

Our preliminary assessment suggests these settlements are within the acceptable tolerance for a residential structure. However, further ground truthing investigations are recommended, particularly in the western portion of the site. This also depends on what is proposed for the earthworks and development program.

7 Conclusion

Based on our observations and analyses, we consider the primary geohazards are:

- Surface deformation as a result of seismically induced liquefaction.

- Potential for lateral spreading from the unsupported free faces in the location of the remnant waterway and towards buried streams.

- Flooding in certain portions of the site.

- Possibility of compressible soils which pose a consolidation risk in certain portions of the site.

In our opinion, the risks presented by these geohazards can be mitigated through industry standard practices found in the Canterbury Region. We therefore do not consider that these hazards should preclude this site from progressing through plan change, however, additional work will be required during the Subdivision Consent phase once the location of buildings and extent of earthworks is known.

Specific to the Canterbury Region, we recommend that the development area is treated as TC2 type land with the relevant design considerations applied as necessary.

We also recommend that the flooding hazard (noted in Section 3.2.4) is considered during civil engineering subdivision design.

8 Future Works

We consider further geotechnical assessment works will be required as the project progresses into subdivision consent and the works may include:

- Undertaking additional geotechnical testing across the site to further delineate the boundaries between TC1 and TC2 type performance and the areas through which compressible material may be present. At a minimum these investigations should include machine boreholes to ground truth the areas with potentially compressible soils and to identify the extent of the gravel deposits encountered in the investigations to date.
- Further assessment of consolidation settlement may need to be completed in the western portion of the site during subdivision design. This is dependent on further ground truthing, and the earthworks and development proposed.
- Dependent on the development and earthworks plan lateral spreading analysis may be required in the vicinity of the unsupported slopes related to the remnant waterway in the east of the site. This can be completed during the subdivision design phase once these elements are known.

9 Sustainability

We encourage you to consider sustainability when assessing the options available for your project. Where suitable for the project, we recommend prioritising the use of sustainable building materials (such as timber in favour of concrete or steel), locally sourced (materials readily available to Contractors as opposed to materials requiring import), and installed in an environmentally friendly way (e.g., reduced carbon emissions and minimal contamination). If you would like to discuss these options further, ENGEO staff are available to offer suggestions.

Site won material should be used wherever possible to minimise the environmental impact of the project. Where site won materials are unsuitable first consider mechanical stabilisation of the fill as an alternative to imported material, and when imported material is still required, consider using recycled aggregates such as crushed concrete – which can absorb carbon out of the atmosphere due to the carbonation process.

10 Limitations

i. We have prepared this report in accordance with the brief as provided. This report has been prepared for the use of our client, Urban Estates Limited, their professional advisers and the relevant Territorial Authorities in relation to the specified project brief described in this report. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity.

ii. The recommendations in this report are based on the ground conditions indicated from published sources, site assessments and subsurface investigations described in this report based on accepted normal methods of site investigations. Only a limited amount of information has been collected to meet the specific 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 should be appreciated that actual conditions could vary from the assumed model.

iii. Subsurface conditions relevant to construction works should be assessed by contractors who can make their own interpretation of the factual data provided. They should perform any additional tests as necessary for their own purposes.

iv. This Limitation should be read in conjunction with the Engineering NZ/ACENZ Standard Terms of Engagement.

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

We trust that this information meets your current requirements. Please do not hesitate to contact the undersigned on (03) 328 9012 if you require any further information.

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11 References

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We also acknowledge the New Zealand GeoNet project and its sponsors EQC, GNS Science and LINZ, for providing data used in this report.

FIGURES

APPENDIX 1:

Geotechnical Testing and Features Map

APPENDIX 2:
Test Pit Logs

APPENDIX 3:

CPT Logs

APPENDIX 4:

Liquefaction Analysis / LSN Analysis