# Table of contents

1 Introduction  
2 Geotechnical investigation  
   2.1 Geology  
   2.2 Previous investigations  
   2.3 Current investigations  
      2.3.1 Machine drilled boreholes  
      2.3.2 Cone Penetration Tests  
      2.3.3 Test Pits  
      2.3.4 Geophysical survey (MASW)  
      2.3.5 Laboratory testing  
      2.3.6 Piezometers  
2.4 Geotechnical model  
3 Liquefaction assessment  
   3.1 Post earthquake observations  
   3.2 Earthquake scenarios  
   3.3 Liquefaction analysis  
   3.4 Lateral spreading  
   3.5 Liquefaction assessment summary  
4 Recommendations for development  
   4.1 Building foundations  
   4.2 Buried infrastructure  
5 Applicability  

Appendix A: Figures  
Appendix B: Borehole logs and photographs  
Appendix C: Cone Penetration Test results  
Appendix D: Test pit logs  
Appendix E: Geophysical survey results  
Appendix F: Laboratory test results  
Appendix G: Liquefaction analysis results
Executive summary

A geotechnical assessment has been conducted by Tonkin & Taylor Ltd for Ngai Tahu Property Ltd at Wigram Skies subdivision in Wigram, Christchurch. The assessment comprised:

- Site walkover assessments to observe any land damage due to liquefaction at the site following the recent earthquakes,
- A geotechnical investigation comprising 24 machine drilled boreholes, 58 Cone Penetration Tests (CPTs), 5km of MASW geophysical survey, 17 test pits and laboratory testing,
- A detailed assessment of liquefaction and lateral spreading hazard at the site, and
- An assessment of foundation options for the houses and recommendations for buried infrastructure.

The results of the investigation and assessment are summarised below:

Geotechnical investigation results

The site can be divided into two zones based on the underlying geotechnical conditions:

- Near surface gravel (located throughout the central portion of the site) – consisting of fine to coarse grained gravels which are dense to very dense. These gravels are indistinguishable from the Riccarton Gravel at greater depths.
- Overbank deposits (located on either side of the near surface gravel) – consisting of interbedded silt and sand layers overlying Riccarton Gravel. The sand layers are typically fine to medium grained and loose to medium dense. The silt layers are low plasticity and range in consistency from soft to firm.

The locations of these zones are provided in Figure A7 in Appendix A.

Groundwater levels varied from 3 to 6m below ground level.

Liquefaction and lateral spreading assessment

No signs of liquefaction damage were observed on the ground surface following the recent earthquakes.

Conclusions from the liquefaction analysis include:

- Non-liquefiable gravels are present over a large portion of the site – therefore the near surface gravel deposits shown in Figure A7 in Appendix A have a very low risk of liquefaction induced ground damage.
- In areas where layers of liquefiable soils are present within the overbank deposits there is a 3.5-6m thick non-liquefiable crust which in most cases is expected prevent liquefaction induced land damage from occurring.
- There are exceptions to this, including an area where a liquefiable sand layer is present from 4-6m below ground level, and small areas adjacent to stormwater detention basins where lateral spreading may occur. These areas are shown in Figure A8 in Appendix A.

No liquefaction induced land damage or lateral spreading is expected in moderate sized earthquakes (1/25 year SLS events). Land damage is only expected to potentially occur in the areas identified above in large earthquakes with return periods greater than 350 years.
Technical categories

The Department of Building and Housing (DBH) has assigned Technical Categories (TCs) to existing houses on the flat land in Christchurch. These categories represent the following levels of liquefaction risk, and different foundation details are recommended for each category:

- TC1 – liquefaction damage unlikely
- TC2 – minor to moderate liquefaction damage is possible in large earthquakes
- TC3 – significant liquefaction damage is possible in large earthquakes

For new sections, technical categories are determined based on the geotechnical investigation and liquefaction assessment. Most of Wigram Skies is classified as TC1, with small areas classified as TC2 (see Figure A8 in Appendix A).

The definition of TC2 encompasses a considerable variation in liquefaction risk. At the areas zoned TC2 within Wigram Skies, liquefaction induced damage is only expected to occur in large earthquakes, and even then the level of damage is expected to be minor. This is because a non-liquefiable ‘crust’ of soil at least four metres deep is present. The combination of this non-liquefiable crust, and the strengthened TC2- type foundations, means that significant damage to houses due to liquefaction is very unlikely.

For the purposes of the liquefaction assessment, ‘large earthquakes’ are defined as earthquakes with a ‘return period’ greater than 350 years. For earthquakes less severe than this no liquefaction damage is expected. To give some perspective, a ‘large earthquake’ (as defined by the post- earthquakes DBH guidelines) is more severe than what occurred at Wigram Skies in all the earthquakes that have occurred in the past two years, and is also more severe than the shaking that is predicted to occur at Wigram Skies due to an Alpine Fault earthquake.

It is important to note that the method used to assign Technical Categories for new sections is very different from the method DBH used to classify existing houses following the earthquakes. For new sections the bar is set much higher – this was done intentionally to reduce economic losses in future earthquakes.

Building foundations

For most of the site conventional shallow foundations such as timber piles or concrete slabs are adequate for houses. These foundation types are consistent with Department of Building and Housing (DBH) guidelines for new house construction in Technical Category 1 areas.

The liquefaction analysis identified areas where due to the liquefaction or lateral spreading risk foundations need to be more robust than those in the low liquefaction risk areas. In these areas, building foundations may achieve adequate performance with a thick concrete slab, a beam grid and slab, a stiffened waffle slab or pile foundations consistent the DBH guidelines for land classified as Technical Category 2 (TC2).

For houses in Lateral Spreading Zone A (shown in Figure A8 in Appendix A), the expected lateral spreading displacements exceed guidelines for TC2 style foundations. To mitigate this risk, T&T has undertaken specific engineering design of a geogrid reinforced gravel raft for these properties, which acts to minimise the expected lateral strain across the house footprint to within the recommended range for land categorised as TC2. This reinforced raft should be constructed beneath all building platforms in lateral spreading Zone A. With this reinforced raft in place, houses in Zone A may be built using TC2 style foundations.
1 Introduction

Tonkin & Taylor Ltd (T&T) has been engaged by Ngai Tahu Property Ltd to undertake a comprehensive geotechnical investigation and liquefaction assessment of Wigram Skies Subdivision in Wigram, Christchurch. This report summarises the work carried out by T&T; describing the geotechnical investigations undertaken and the identified ground conditions, an assessment of liquefaction risk, and recommendations for house foundations and infrastructure.

Wigram Skies subdivision is situated on the site of the former Wigram Aerodrome. The project consists of approximately 1000 lots with a total area of approximately 120ha.

T&T have undertaken the following scope of work for the purposes of this report:

- A site specific geotechnical investigation,
- A detailed liquefaction analysis, and,
- Preliminary foundation assessment for commercial and industrial buildings on the site.

The work has been completed in accordance with the terms and conditions outlined in T&T’s letter of engagement dated 28 February 2012. This report was first issued in July 2012, this version of the report is to update the geotechnical assessment following changes to the subdivision layout and to incorporate guidance released by the Ministry of Business, Innovation and Employment (MBIE) in December 2012.

Section 2 of this report summarises the geotechnical investigations, Section 3 summarises the results of the liquefaction assessment and Section 4 describes the foundation assessment and contains recommendations for the subdivision development.
2 Geotechnical investigation

2.1 Geology

Geological maps of the area describe the near surface soils as the Yaldhurst Member of the Springston Formation. The Springston Formation comprises post-glacial river channel and flood plain sediments; the Yaldhurst Member was deposited during the last 3000 years.

Figure 1 is a plan view of the subdivision with mapped geological units overlain on top. The dark yellow colour indicates gravels deposited from historic flood channels of the Waimakariri River. The light yellow colour indicates ‘overbank’ deposits, which have been deposited during floods that have occurred throughout the past few thousands of years. During a flood event the larger sand particles settle first, following by the finer silt particles settling on top. The result of this complex depositional history is a highly layered and variable soil profile.

Beneath these layers at depths greater than 13 – 14m is the ‘Riccarton Gravel’ deposited during the most recent glacial period (14 000 – 70 000 years ago).

2.2 Previous investigations

A number of previous investigations have been undertaken at the site:

- Test pits undertaken by T&T for a contaminated land assessment in October 2008
- Test pits undertaken by Eliot Sinclair in October 2010
- Five deep boreholes, four Cone Penetration Tests (CPTs) and geophysical survey (MASW) undertaken by T&T for a Wigram Skies liquefaction assessment in February 2011

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Ten machine drilled boreholes and ten CPTs undertaken by T&T for a geotechnical investigation in November and December 2011 at the Wigram Skies Business Park site, to the north-east of the subdivision.

Four piezometers on the site between 18 January 2011 and 21 May 2012 to monitor ground water depths.

These investigations confirmed the overall geological structure is similar to the geological map; with the exception that the extent of the near surface gravel area is more extensive.

2.3 Current investigations

A programme of additional ground investigations was undertaken to provide a comprehensive geotechnical investigation across the entire Wigram Skies subdivision. The investigation was intended to follow guidelines recently published\(^2\) by the Ministry of Business, Innovation and Employment (MBIE) for the development of new subdivisions.

2.3.1 Machine drilled boreholes

Twenty four machine drilled boreholes were carried out on site from 13 April to 04 May 2012 by Prodrill Ltd using a sonic rotary drill rig. The boreholes were carried out to a minimum depth of 15.45m and a maximum depth of 21.45 m below ground level.

The borehole samples were logged in accordance with the NZ Geotechnical Society Guidelines. Standard Penetration Tests (SPTs) were undertaken at 1.5m intervals.

The borehole locations are shown on Figure A1 in Appendix A; the borehole logs are presented in Appendix B.

A large number of boreholes in the centre of the site showed dense to very dense gravel from near the ground surface to depths greater than 15m below ground level. On the eastern and western sides of the site the boreholes encountered interbedded silt and sand layers overlying gravel. Table 1 summarises the soil layers encountered in the investigation. The depths and thickness of the sand and silt layers vary considerably across the site; see the geological sections in Figures A2 to A6 in Appendix A.

Table 1 – Borehole summary

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>Engineering description</th>
<th>SPT N value</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAND layers</td>
<td>SAND with minor/some silt, fine grained, loose to medium dense</td>
<td>N = 2 to 15</td>
</tr>
<tr>
<td>SILT layers</td>
<td>SILT with some sand, non plasticity to low plasticity, firm to soft</td>
<td>N = 2 to 10</td>
</tr>
<tr>
<td>GRAVEL layers</td>
<td>GRAVEL with some sand and silt, fine to coarse grained, dense to very dense</td>
<td>N = 20 to 50+</td>
</tr>
</tbody>
</table>

2.3.2 Cone Penetration Tests

58 Cone Penetration Tests (CPTs) were carried out on site between 18 April and 01 June 2012. by Opus International Consultants Ltd, McMillan Drilling Services and Lankelma Ltd. The CPTs were performed to refusal with a maximum depth of 15m below ground level.

\(^2\) Ministry of Business, Innovation & Employment – Repairing and rebuilding houses affected by the Canterbury Earthquake sequence – Part D: Guidelines for the geotechnical investigation and assessment of subdivisions in the Canterbury region, December 2012
The CPT tests were performed at the locations shown on Figure A1 in Appendix A, and results are shown in Appendix B. The depths and thickness of the sand and silt layers vary considerably across the site; see the geological sections in Figures A2 to A6 in Appendix A.

Table 2 – CPT results summary

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>CPT tip resistance, qc (MPa)</th>
<th>CPT friction ratio, Rf (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAND layers</td>
<td>3 – 6 MPa</td>
<td>0.5 – 1 %</td>
</tr>
<tr>
<td>SILT layers</td>
<td>0.5 – 1.0 MPa</td>
<td>2 – 4 %</td>
</tr>
<tr>
<td>GRAVEL layers</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

2.3.3 Test Pits

17 test pits were excavated on the site on the 16 and 17 May 2012 to a maximum depth of 3.0 m below ground level. The test pits were excavated in the areas of the site where near surface gravels were present, and were intended to prove the surface of the dense gravel layer.

The test pits were supervised and logged by a geotechnical engineer from T&T. The locations of the test pits are shown in Figure A1 in Appendix A and the test pit logs are presented in Appendix D.

2.3.4 Geophysical survey (MASW)

5km of geophysical survey using the Multichannel Analysis of Surface Waves (MASW) technique was carried out by Southern Geophysical Ltd between 12 and 23 April 2012. A number of lines were surveyed across the site; these survey lines provide a two dimensional view of the shear wave velocity of the soil versus depth. The shear wave velocity can be used to determine the soil characteristics, including soil density and liquefaction resistance.

The MASW results were calibrated to the borehole and CPT results, and then used to interpolate between the other investigation locations to produce a continuous soil profile across the site.

The locations of the MASW survey lines are shown in Figure A1 in Appendix A, with the output presented in Appendix E.

2.3.5 Laboratory testing

The following laboratory testing was undertaken by Geotechnics Ltd on samples recovered from the boreholes:

- 29 fines content tests on the sandy silt and silty sand samples
- 22 Atterberg Limit tests on silt samples from 2.6 to 11.5 m below ground level.

A summary of the laboratory test results is presented in Table 3, Figure 2 and in Appendix F.
### Table 3 – Laboratory testing results

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Depth</th>
<th>Natural water content, WC (%)</th>
<th>Liquid Limit, LL (%)</th>
<th>Plastic Limit, PL (%)</th>
<th>Plasticity Index, PI</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.3-3.1</td>
<td>39.3</td>
<td>37</td>
<td>27</td>
<td>10</td>
</tr>
<tr>
<td>1</td>
<td>4.6-4.7</td>
<td>41.3</td>
<td>41</td>
<td>31</td>
<td>10</td>
</tr>
<tr>
<td>1</td>
<td>7.1-7.2</td>
<td>89.8</td>
<td>89</td>
<td>56</td>
<td>33</td>
</tr>
<tr>
<td>2</td>
<td>8.3-8.4</td>
<td>25.1</td>
<td>30</td>
<td>23</td>
<td>7</td>
</tr>
<tr>
<td>2</td>
<td>10.5-10.6</td>
<td>41.7</td>
<td>40</td>
<td>32</td>
<td>8</td>
</tr>
<tr>
<td>3</td>
<td>3.7-3.8</td>
<td>98.9</td>
<td>65</td>
<td>51</td>
<td>14</td>
</tr>
<tr>
<td>3</td>
<td>9.5-9.6</td>
<td>31.4</td>
<td>45</td>
<td>35</td>
<td>10</td>
</tr>
<tr>
<td>4</td>
<td>3.5-3.6</td>
<td>56.5</td>
<td>33</td>
<td>26</td>
<td>7</td>
</tr>
<tr>
<td>7</td>
<td>2.6-2.7</td>
<td>28</td>
<td>29</td>
<td>24</td>
<td>5</td>
</tr>
<tr>
<td>15</td>
<td>5.1-5.3</td>
<td>62.8</td>
<td>52</td>
<td>41</td>
<td>11</td>
</tr>
<tr>
<td>17</td>
<td>3.7-3.8</td>
<td>29.5</td>
<td>29</td>
<td>21</td>
<td>8</td>
</tr>
<tr>
<td>17</td>
<td>6.5-6.6</td>
<td>43.5</td>
<td>38</td>
<td>29</td>
<td>9</td>
</tr>
<tr>
<td>20</td>
<td>3.5-3.6</td>
<td>32.2</td>
<td>31</td>
<td>24</td>
<td>7</td>
</tr>
<tr>
<td>20</td>
<td>9.5-9.6</td>
<td>44.9</td>
<td>46</td>
<td>33</td>
<td>13</td>
</tr>
<tr>
<td>21</td>
<td>7.1-7.2</td>
<td>42.7</td>
<td>35</td>
<td>27</td>
<td>8</td>
</tr>
<tr>
<td>22</td>
<td>7.1-7.2</td>
<td>37</td>
<td>48</td>
<td>30</td>
<td>18</td>
</tr>
<tr>
<td>22</td>
<td>10.6-10.7</td>
<td>52</td>
<td>71</td>
<td>44</td>
<td>27</td>
</tr>
<tr>
<td>23</td>
<td>5.8-5.9</td>
<td>26.3</td>
<td>28</td>
<td>21</td>
<td>7</td>
</tr>
</tbody>
</table>

**Figure 2** – Unified classification chart summarising laboratory testing results in the SILT layers across the site – the silt layers are predominately low plasticity silts
2.3.6 Piezometers

A number of piezometers were installed across the site to monitor ground water levels. The piezometers were screened from 3 – 6m below ground level. Table 4 summarises the water levels observed during monitoring.

Table 4 – Summary of piezometer results

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Elevation (m RL)</th>
<th>Groundwater elevation (m RL) @ 17 May 2012</th>
<th>Groundwater depth @ 17 May 2012</th>
<th>Groundwater elevation (m RL) @ 11 July 2012</th>
<th>Groundwater depth @ 11 July 2012</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH02 (2012)</td>
<td>31.81</td>
<td>25.9</td>
<td>5.9</td>
<td>25.8</td>
<td>6.0</td>
</tr>
<tr>
<td>BH04 (2012)</td>
<td>30.17</td>
<td>25.3</td>
<td>4.9</td>
<td>25.4</td>
<td>4.8</td>
</tr>
<tr>
<td>BH11 (2012)</td>
<td>29.48</td>
<td>23.9</td>
<td>5.6</td>
<td>23.9</td>
<td>5.6</td>
</tr>
<tr>
<td>BH20 (2012)</td>
<td>28.23</td>
<td>24.6</td>
<td>3.6</td>
<td>24.4</td>
<td>3.8</td>
</tr>
<tr>
<td>BH01 (2010)</td>
<td>30.66</td>
<td>28.6*</td>
<td>3.2*</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>BH02 (2010)</td>
<td>31.81</td>
<td>24.0</td>
<td>4.7</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>BH03 (2010)</td>
<td>30.51</td>
<td>23.5</td>
<td>5.0</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>BH04 (2010)</td>
<td>30</td>
<td>24.5</td>
<td>5.5</td>
<td>**</td>
<td>**</td>
</tr>
</tbody>
</table>

*This piezometer reading is not consistent with nearby piezometers, ECan well logs, test pits and the dry base of the nearby Awatea Stormwater basins. This discrepancy is attributed to inadequate grouting at the base of the piezometer, resulting in upward flow of groundwater from the underlying gravel aquifer causing an artificially high groundwater reading.

**These piezometers have been destroyed during subdivision earthworks

2.4 Geotechnical model

The site can be divided into two zones based on the underlying geotechnical conditions:

- Near surface gravel – consisting of fine to coarse grained gravels which are dense to very dense. These gravels are indistinguishable from the Riccarton Gravel at greater depths.
- Overbank deposits – consisting of interbedded silt and sand layers overlying Riccarton Gravel. The sand layers are typically fine to medium grained and loose to medium dense. The silt layers are low plasticity and range in consistency from soft to firm.

The location of these zones is provided in Figure A7 in Appendix A. Geological cross sections are provided in Figures A2 to A6 which show the extent of the sand and silt layers within the overbank deposits.
3 Liquefaction assessment

3.1 Post earthquake observations

Seismic liquefaction occurs when excess pore pressures are generated in loose, saturated, generally cohesionless soil during earthquake shaking, causing the soil to undergo a partial to complete loss of strength. The occurrence of liquefaction is dependent on several factors, including the intensity and duration of ground shaking, soil density, particle size distribution, and elevation of the groundwater table.

Following the recent earthquakes, site inspections were undertaken by engineers from T&T to observe if any ground damage occurred as a result of liquefaction. No earthquake or liquefaction induced damage was observed.

Table 5 summarises ground motions recorded nearby from four significant earthquakes that have affected Christchurch during the previous two years.

Table 5 – Summary of post earthquake observations

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Moment Magnitude (Mw)</th>
<th>Estimated peak ground acceleration (PGA) at the site*</th>
<th>Estimated peak ground acceleration (PGA) scaled to Mw=7.5 earthquake**</th>
<th>Observed damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 September 2010</td>
<td>7.1</td>
<td>0.23g</td>
<td>0.21g</td>
<td>No observed damage</td>
</tr>
<tr>
<td>22 February 2011</td>
<td>6.2</td>
<td>0.30g</td>
<td>0.21g</td>
<td>No observed damage</td>
</tr>
<tr>
<td>13 June 2011</td>
<td>6.0</td>
<td>0.21g</td>
<td>0.14g</td>
<td>No observed damage</td>
</tr>
<tr>
<td>23 December 2011</td>
<td>5.9</td>
<td>0.20g</td>
<td>0.13g</td>
<td>No observed damage</td>
</tr>
</tbody>
</table>

* Estimated from recorded peak ground acceleration at Riccarton High School (RHSC) seismograph, located 1.5km north of the site
** Using magnitude scaling factor from Idriss and Boulanger (2008)

3.2 Earthquake scenarios

Our liquefaction assessment considers two earthquake scenarios derived from NZS1170.5 and considering guidance on ground shaking hazard in Canterbury from the MBIE’s Engineering Advisory Group3.

These scenarios are summarised in Table 6 below, and represent the following design performance requirements:

- Serviceability Limit State (SLS) – 25 year return period event – to avoid damage that would prevent buildings from being used as originally intended without repair; and
- Ultimate Limit State (ULS) – 500 year return period event – to avoid collapse of buildings and protect life.

A site soil class of D (deep or soft soils) was assumed due to the large depth to bedrock in the Christchurch area (greater than 500m).

Table 6 – Summary of earthquake scenarios used in the liquefaction assessment

<table>
<thead>
<tr>
<th></th>
<th>Serviceability Limit State (SLS1)</th>
<th>Ultimate limit state (ULS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Return period (years)</td>
<td>25</td>
<td>500</td>
</tr>
<tr>
<td>Magnitude, M</td>
<td>7.5</td>
<td>7.5</td>
</tr>
<tr>
<td>Peak horizontal ground acceleration, PGA</td>
<td>0.13g</td>
<td>0.35g</td>
</tr>
</tbody>
</table>

Based on the information in Table 5 and Table 6, the ground accelerations experienced at the site on the 4 September 2010 and 22 February 2011 earthquakes are judged to be less than ULS and greater than SLS design level values, especially when magnitude effects are considered (a higher earthquake magnitude implies a longer duration of shaking). The 13 June 2011 and 23 December 2011 ground motions were slightly less than the SLS design level.

### 3.3 Liquefaction analysis

Analyses were performed to evaluate the liquefaction potential of the soil layers using the methods recommended by Idriss and Boulanger (2008). The two earthquake scenarios described in Table 6 were used as input ground motions and the ground water level was assumed based on the piezometer readings. The liquefaction analysis outputs are presented in Appendix F.

On the western side of the site, the ground elevation varies from 30.0 – 32.0m RL, with the elevation of the water table varying from 24.0 – 25.9m RL. A water table depth of 4m was assumed in the liquefaction analysis in this area. On the eastern side of the site, the ground elevation varies from 28.2 – 29.1m RL with the elevation of the water table varying from 23.5 – 24.6m RL. A water table depth of 3.5m was assumed in the liquefaction analysis in this area. These water table depths are likely to be conservative and represent unusually high groundwater conditions, i.e. the water table is likely to be lower than this during normal conditions.

The results of the liquefaction analysis indicate that:

- The SAND layers are considered liquefiable in both the SLS and ULS events.
- The GRAVEL layers are considered too permeable and often too dense to liquefy.
- The SILT layers are also considered non-liquefiable. This is based on the CPT results and laboratory testing which indicated that the SILT typically has a plasticity index of PI = 7 to 15; international research and experience following the Christchurch earthquakes indicates these soils are not liquefiable.

The liquefaction analysis results were then processed using the method of Ishihara (1985), which compares the thickness of liquefiable layers with the thickness of the non-liquefiable crust material. Figure 3 shows the results of this method for our two design cases, the SLS and ULS earthquakes. If a point plots above the line, liquefaction induced ground damage is expected, if below then no damage is expected.

It can be seen that:

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• No liquefaction is expected to occur in the central portion of the site which is underlain by shallow gravel.
• While liquefaction of some layers within the overbank deposits is expected the thickness of the non-liquefiable crust is expected to prevent any damage occurring on the ground surface in the SLS event.
• No liquefaction induced ground damage is expected in the majority of locations within the overbank deposits during an ULS earthquake event.
• A small portion of investigation locations indicate that liquefaction induced land damage can be expected in an ULS earthquake. These locations correspond to an area of the site where a 2-3m thick liquefiable sand layer is present between 4 and 7m below ground level. It is estimated that a large earthquake with a return period of 350 years or greater would be required to cause damage in these areas.

Note that this assessment has been made using groundwater levels which are generally shallower than that observed in the piezometers; therefore this assessment is likely to be conservative.

The magnitude of liquefaction induced ground settlements was evaluated using the method of Zhang, Robertson and Brachman (2002). This analysis indicated that in the overbank deposits settlements of 50-100mm can be expected in a SLS event and 100-200mm can be expected in an ULS event. No settlement is expected in the near surface gravel area. These settlements are theoretical however, and recent experience in Christchurch indicates that the theoretical settlements have not been observed in areas such as Wigram with a 3-4m thick non-liquefied crust and where liquefiable layers are interbedded with non-liquefiable layers.

In addition, no ground settlement has been observed in surveys undertaken by Eliot Sinclair following the recent earthquakes despite the Zhang, Robertson and Brachman method predicting 100-200mm of settlement following the 4 September 2011 and 22 February 2011 earthquakes.

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Figure 3 – Comparison of non-liquefied crust thickness and liquefied layer thickness for (a) SLS earthquake event, and (b) ULS earthquake event. Points plotting above the solid line indicate that liquefaction induced ground damage is expected; points below the line indicate that liquefaction induced ground damage is not expected.
3.4 Lateral spreading

Lateral spreading is a phenomenon associated with liquefaction, where large horizontal ground displacements can occur on land adjacent to rivers, streams, wetlands or on gentle slopes.

As the site is flat, lateral spreading can only occur adjacent to man-made features such as stormwater detention basins, drains and diverted streams. These include:

- Stormwater detention basins SW1, SW2 and SW3 along the eastern boundary of the site (see Figure A8 in Appendix A). These are associated with a returning Hayton Stream to its former course. The depth of the stream varies from 1.5m to the north to 2.5m to the south.
- Stormwater detention basin SW4 on the southern boundary of the site adjacent to Wigram Road (see Figure A8 in Appendix A). This basin comprises a first flush basin (depth varies from 1.2 – 2.3m) and a detention basin (depth varies from 2.1 – 2.8m).
- Stormwater detention basin along Awatea Road on the western boundary of the site (see Figure A8 in Appendix A). The depth of these basins varies from 3 – 4m.
- Swales at various locations across the subdivision which vary in depth from 1-2m.

Lateral spreading is expected in areas are where a potentially liquefiable layer is present within 2H of the ground surface, where H = the basin depth. Outside of these areas, damaging lateral spreading is not expected to occur.

Factors affecting the occurrence and severity of lateral spreading include the basin depth, the thickness of the liquefiable layer, and the intensity of ground shaking. These factors were considered using the methods of Jibson (2007)\(^7\) and Zhang et al. (2004)\(^8\) to determine the magnitude and extent of expected lateral spreading displacements.

For areas where a lateral spreading risk was identified the calculations were used to identify zones (Zone A and Zone B) near the stormwater basins as shown on Figure A8 in Appendix A. Zone A represents an area of moderate lateral spreading risk, and Zone B represents an area of low lateral spreading risk. Recommended foundations to mitigate the lateral spreading risk are discussed in Section 4.1.

3.5 Liquefaction assessment summary

When considering the potential for liquefaction induced damage at Wigram Skies the following conclusions can be made:

- Non-liquefiable gravels are present over a large portion of the site.
- In areas where layers of liquefiable soils are present there is a 3.5-6m thick non-liquefiable crust which in most cases is expected prevent liquefaction induced land damage from occurring.
- There are exceptions to this, including an area where a liquefiable sand layer is present from 4-6m below ground level, and small areas adjacent to stormwater detention basins where lateral spreading may occur. These areas are shown in Figure A8 in Appendix A.


No liquefaction induced land damage or lateral spreading is expected in moderate sized earthquakes (1/25 year SLS events). Damage is only expected in the areas identified above in large earthquakes which return periods greater than 350 years.
4 Recommendations for development

4.1 Building foundations

The results of the liquefaction assessment indicate that there is a low risk of liquefaction induced land damage over the majority of the subdivision, with the exception of the areas identified in Figure A8 in Appendix A.

Therefore for all lots (except for these areas identified in Figure A8 in Appendix A), provided the shallow investigations (i.e. scalas and hand augers) undertaken on each lot indicate that the geotechnical bearing capacity is greater than 300kPa, then foundations such timber floors with piles, or timber floors with a perimeter concrete footing can be constructed in accordance with NZS 3604. Concrete slab on grade foundations can also be constructed in accordance with NZS 3604 including modification B1/AS1, which requires ductile reinforcing in the floor slabs. These foundation recommendations are consistent with areas classified as Technical Category 1 by MBIE.

In the lateral spreading Zone B and areas where a liquefiable layer is present from 4-6m below ground level building foundations may achieve adequate performance with a thick concrete slab, a beam grid and slab, a stiffened waffle slab or pile foundations consistent with Foundation Options 2 to 5 described in Section 5.3 of the MBIE guidelines. For Foundation Options 2 - 4 a double-layer of polythene is recommended between the ground and the underside of the slab. The recommendations in the lateral spreading zones B are consistent with MBIE recommendations for land classified as Technical Category 2.

In lateral spreading Zone A, the analysis suggests that if no site improvement works were undertaken then a very large ULS earthquake may cause moderate lateral ground displacements which exceed the limits that have been defined for TC2-type foundations. To mitigate this risk, T&T has undertaken specific engineering design of a geogrid reinforced gravel raft for these properties, which acts to minimise the expected lateral strain across the house footprint to within the recommended range for land categorised as TC2. The gravel raft consists of:

- A raft of granular hardfill such as gravel or recycled crushed concrete compacted in layers that extend one metre below the foundation depth of the shallow footing and a minimum of 1.5 metres around the building perimeter to provide a solid, non-liquefiable foundation subgrade.
- Four layers of geogrid reinforcement in the hardfill using a biaxial grid such as SS20 or similar, to tie the hardfill raft together to minimise ground extension and cracking beneath the foundations.
- Polythene layers to provide a slip plane between the concrete slab and gravel raft, to limit the forces which can be transferred to the slab by horizontal movement of the underlying ground.

This reinforced raft should be constructed beneath all building platforms in lateral spreading Zone A. With this reinforced raft in place, houses in Zone A may be built using the same foundation options outlined above for Zone B. Figure A9 in Appendix A shows the details of the raft to be constructed.

4.2 Buried infrastructure

Significant liquefaction induced ground damage is not expected in an ULS event across the subdivision. Lateral spreading displacements are only expected to occur in lateral spreading zones A and B (as shown in Figure A8 in Appendix A) during large earthquakes (i.e. return periods
greater than 350 years). Given this, specific mitigation works to protect buried infrastructure are not considered necessary.

In general, it is expected that buried services such as manhole risers, pump station chambers and pipes will be founded above the water table. Where services are to be founded below the water table, it is recommended that the service trenches be backfilled with free draining gravel, lined with filter fabric (such as Bidim A19 or similar) and designed to withstand uplift pressures and differential settlements. Flexible pipes and connections should be used where practicable.
5 Applicability

This report has been prepared for the benefit of Ngai Tahu Property Ltd with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

Tonkin & Taylor Ltd
Environmental and Engineering Consultants

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Geotechnical Engineer

Authorised for Tonkin & Taylor Ltd by: Peter Millar
Project Director

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Appendix A: Figures

- Figure A1 – Investigation Location Plan
- Figure A2 – A6 – Geological Cross Sections
- Figure A7 – Geological zones
- Figure A8 – Building foundation zones
LEGEND

Silt dominant horizon, typically silt with some sand, low plasticity, soft to firm.

Gravel, fine to coarse grained, dense to very dense.

Sand dominant horizon, typically fine to medium grained sand, loose to medium dense.

Sand
Silt
Gravel

Reduced level (m)

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Figure A2
Reduced level (m)

Gravel
Silt
Sand

Topsoil

LEGEND

Sand dominant horizon, typically fine to medium grained sand, loose to medium dense.
Silt dominant horizon, typically silt with some sand, low plasticity, soft to firm.
Gravel, fine to coarse grained, dense to very dense.
LEGEND

- **Topsoil**: Sand dominant horizon, typically fine to medium grained sand, loose to medium dense.
- **Silt**: Silt dominant horizon, typically silt with some sand, low plasticity, soft to firm.
- **Gravel**: Gravel, fine to coarse grained, dense to very dense.

Reduced level (m)

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LEGEND

Topsoil  Sand dominant horizon, typically fine to medium grained sand, loose to medium dense.
Silt  Silt dominant horizon, typically silt with some sand, low plasticity, soft to firm.
Gravel  Gravel, fine to coarse grained, dense to very dense.
LEGEND

Sand dominant horizon, typically fine to medium grained sand, loose to medium dense.

Silt dominant horizon, typically silt with some sand, low plasticity, soft to firm.

Gravel, fine to coarse grained, dense to very dense.