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Ngai Tahu Property Ltd
PO Box 130060
Christchurch

Attention: Alan Grove

Dear Alan

Wigram Skies Subdivision
Stages 2A and 2B Geotechnical Assessment

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 letter summarises the work carried out by T&T in Stages 2A and 2B of the subdivision; describing the geotechnical investigations undertaken and the identified ground conditions, an assessment of liquefaction risk, and recommendations for house foundations and infrastructure.

1. **Geotechnical investigations**

The geotechnical investigation comprised:

- Four machine drilled boreholes to a maximum depth of 20m below ground level, with Standard Penetration Tests (SPTs) at 1.5m intervals,
- 18 Cone Penetration Tests (CPTs) to a maximum depth of 14m below ground level,
- 700m of geophysical survey (MASW) to a depth of 20m below ground level, and,
- Laboratory testing of borehole samples.

The borehole logs, CPT and MASW results will be presented in a final report as part of a subdivision wide geotechnical assessment.

2. **Ground conditions**

The soil profile at the site consists of interbedded sand and silt layers overlying gravel. The depth to gravel varies across Stage 2; to the west (Stage 2A) it is 13 – 14m below ground level; for Stage 2B it is typically 7 – 8m. The depth to gravel decreases at the east end of Stage 2B, and eventually the gravel is present at the ground surface.

The sand and silt layers are ‘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. 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 gravel that is close to the surface is deposited from historic flood channels of the Waimakariri River. The gravel at depths greater than 13 – 14m is the ‘Riccarton Gravel’ deposited during the most recent glacial period (14 000 – 70 000 years ago).

3. Liquefaction risk

The risk of liquefaction at the site was assessed using the CPT, SPT and MASW data. Laboratory test results are not yet available. The liquefaction risk was assessed for two earthquake scenarios:

- **Serviceability Limit State (SLS)** – M7.5, PGA=0.13g – this represents a 1/25 year earthquake. Structures are to be designed so that little or no structural damage occurs in this earthquake, and that services remain functional and any damage is readily repairable.
- **Ultimate Limit State (ULS)** – M7.5, PGA=0.35g – this represents a 1/500 year earthquake. Structures are designed to avoid collapse in this level of earthquake shaking.

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).

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 undertaken previously which indicated that the SILT has a plasticity index of PI = 9 to 13. Results of the current laboratory testing will be used to confirm this assessment.

The liquefiable SAND layers are relatively thin, and sandwiched between non-liquefiable silt layers. From observations in the Christchurch earthquakes, liquefaction induced settlements were minor where this was the case, especially when the non-liquefiable crust is sufficiently thick and strong to limit ejection of material.

The liquefaction analysis results were processed using the method of Ishihara (1985), which compares the thickness of liquefiable layers with the thickness of the non-liquefiable crust material. Figure 1 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.

Figure 1 shows that:

- No liquefaction induced ground damage is expected to occur in the SLS earthquake event.
- No liquefaction induced ground damage is expected to occur over the majority of the site during an ULS event. Some parts of the site are on the ‘borderline’ – this indicates that some damage may be expected in some areas, but this damage is expected to be relatively minor and only occur in an extreme earthquake event.
No signs of liquefaction, or liquefaction induced ground damage, were observed following the recent earthquakes. 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 greater than the SLS design level. The liquefaction analysis results are consistent with these observations.

In summary, the liquefiable SAND layers are not expected to cause significant ground damage as they are not very thick and there is a non-liquefiable crust of at least three metres thick, which is expected to limit ejection of underlying liquefied material.

4. Lateral spreading

While the liquefiable SAND layers described above are not expected to cause significant ground damage, there is a risk that liquefaction of these layers will cause lateral spreading to occur adjacent to the Awatea stormater detention basins, located to the south-west of Stage 2A.

No lateral spreading was observed following the recent earthquakes, but there is a risk that it may occur in a future ULS earthquake. A lateral spreading analysis was undertaken to determine the area of land that is potentially at risk, and to determine appropriate measures to mitigate this risk.

Two lateral spreading zones were identified within Stage 2A as shown on Figure A1 in Appendix A. Zone A represents an area of moderate lateral spreading risk, and Zone B represents an area of low lateral spreading risk.

5. Building foundations

The results of the liquefaction assessment indicate that there is a low risk of liquefaction induced land damage in Stages 2A and 2B, with the exception of the lateral spreading zones A and B identified above.
Therefore for all lots in Stages 2A and 2B except for lateral spreading zones A and B, 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 the Department of Building and Housing (DBH).

In the lateral spreading zones building foundations may comprise 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 DBH guidelines ‘Revised guidance on repairing and rebuilding houses affected by the Canterbury earthquake sequence’. For Foundation Options 2 - 4 a double-layer of polythene between the ground and the underside of the slab. The recommendations in the lateral spreading zones B are consistent with DBH recommendations for land classified as Technical Category 2.

In lateral spreading Zone A, analysis suggests that a very large earthquake might 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 lateral strain across the house footprint. Construction of this reinforced raft has now been completed beneath all building platforms in lateral spreading Zone A. With this reinforced raft in place, houses in Zone A can build using the same foundation options outlined above for Zone B. Figure A2 in Appendix A shows the details of the raft which has been constructed.

6. Recommendations for buried infrastructure

Significant liquefaction induced ground damage is not expected in an ULS event across the majority of Stages 2A and 2B. Lateral spreading displacements are only expected to occur in lateral spreading zones A and B (as shown in Figure A1 in Appendix A) during large earthquakes (i.e. return periods greater than 400 years). Given this, specific mitigation works to protect buried infrastructure within Stage 2A and 2B 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.
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
Report prepared by: Hayden Bowen
Geotechnical Engineer

Authorised for Tonkin & Taylor Ltd by: Anthony Fairclough
Project Director
Appendix A: Figures

- Figure A1 – Lateral spreading zones within Stage 2A
- Figure A2 – Details of reinforced gravel hardfill raft
Engineer shall be notified on completion of excavation for services over raft and shall inspect the trench prior to any backfilling or forming to confirm that the geogrid is undamaged.

Notes:
1) Services entry details into side wall of foundation. Details by raft supplier.
2) Sewer and waste pipes to exit foundation through side wall of foundation. Details by raft supplier.
3) Plastic geogrid reinforcement in gravel raft shall not be penetrated or damaged by services installation. Fence post excavations may penetrate the upper layer of geogrid provided the hole diameter does not exceed 250mm.
4) This will require all services to be laid within a maximum of 650mm depth from ground surface to trench invert.
5) Should any geogrid be damaged during excavation the engineer will specify repair procedure.
6) All underfloor services and drains shall be installed within the concrete raft thickness. See raft suppliers details.
7) All connections between external drains and pipes within the concrete raft shall be accessible for replacement should ground movement occur.
8) Foundations shall not encroach closer than 1.5m to edge of gravel raft on all sides. See plan for dimensions to gravel raft.
9) These details are to be read in conjunction with the latest version Department of Building and Housing document (DHB guidelines) "Guidance on house repairs and reconstruction following the Canterbury earthquake".
10) The gravel raft can be extended but any extensions shall be subject to specific design and construction observation by Eliot Sinclair & Partners Ltd.
11) Compacted silt layer & gravel raft shall meet the provisions of NZS 4431 "Code of Practice for Earth Fill for Residential Development".

TYPICAL CROSS-SECTION THROUGH FOUNDATION AND GRAVEL RAFT
Scale 1:50 (A3)

Notes:
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2) Sewer and waste pipes to exit foundation through side wall of foundation. Details by raft supplier.
3) Plastic geogrid reinforcement in gravel raft shall not be penetrated or damaged by services installation. Fence post excavations may penetrate the upper layer of geogrid provided the hole diameter does not exceed 250mm.
4) This will require all services to be laid within a maximum of 650mm depth from ground surface to trench invert.
5) Should any geogrid be damaged during excavation the engineer will specify repair procedure.
6) All underfloor services and drains shall be installed within the concrete raft thickness. See raft suppliers details.
7) All connections between external drains and pipes within the concrete raft shall be accessible for replacement should ground movement occur.
8) Foundations shall not encroach closer than 1.5m to edge of gravel raft on all sides. See plan for dimensions to gravel raft.
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PLAN - TYPICAL LOT & RAFT ARRANGEMENT
Plan view not to scale
(See sheet 1 for dimensions to extent of gravel raft on individual lots)