IN THE MATTER	of the Resource Management Act 1991
AND	
IN THE MATTER	of a submission in respect of the **PROPOSED WAIKATO DISTRICT PLAN** by **AMBURY PROPERTIES LIMITED** pursuant to Clause 6 of Schedule 1 of the Act seeking the rezoning of land at Ohinewai

STATEMENT OF EVIDENCE OF NICHOLAS IAN SPEIGHT

1. INTRODUCTION

1.1 My name is Nicholas Ian Speight. I am a Senior Geotechnical Engineer and Director of Initia Ltd, a specialist geotechnical consultancy company. I have been in this role for 2 years. Prior to this, I was a Senior Geotechnical Engineer and Major Shareholder at Tonkin & Taylor Ltd, an environmental and engineering consultancy firm.

Qualifications and experience

1.2 I hold the degree of Bachelor of Engineering (Hons) in Civil Engineering from the University of Canterbury (1999). I am a Member of Engineering New Zealand and a member of the New Zealand Geotechnical Society Incorporated. I am a Chartered Professional Engineer (CPeng), International Professional Engineer (IntPE), and I have 20 years' post-graduate experience in geotechnical engineering.

1.3 My geotechnical experience has been gained on a wide range of projects including major commercial and residential subdivisions, large infrastructure projects such as Auckland Airport's second runway, and multi-storey commercial and residential buildings, often with deep basements.

1.4 I have provided specialist geotechnical services on many projects in New Zealand. My experience has included investigations into, and design of, ground improvement of greenfield land for commercial and light industrial development.

1.5 In my former role at Tonkin & Taylor I was the geotechnical lead for the development of over 100 ha of greenfield land for Auckland Airport's commercial business park, 'The Landing'. This included managing investigations, design of earthworks and providing geotechnical advice for design and construction of civil infrastructure for the subdivision. I also provided geotechnical advice for design and construction of large-scale, light industrial and commercial buildings such as the new Foodstuffs Distribution Centre at Auckland Airport, with a footprint of over 7.5 ha, and the Sistema warehouse and factory, a 5.5 ha building in Mangere.

1.6 In the Waikato Region, I have provided investigation and ground improvement design advice to Tainui Group Holdings for the concept design of the Ruakura Inland Port and Logistics Hub development. I am therefore familiar with the geotechnical constraints and design issues for large scale, greenfield development situated on challenging geology and soil conditions.

1.7 In my role at Initia, I have provided geotechnical advice to Auckland Airport for Stage 5 of The Landing subdivision, a 20 ha development of greenfield land, which requires ground improvement by preloading. I am also presently involved with no fewer than five large scale warehouse developments in the same area. Until March 2020, I was "Design Manager" for Auckland Airport's proposed second runway. The "Design Manager" role involved the co-ordination and management of international and local design consultants, construction advisors and cost estimators.

Involvement in project

1.8 In 2018, Initia was engaged by Ambury Properties Limited ("APL") to complete a pre-purchase assessment of the site at 52-58 Lumsden Road, 88 Lumsden Road and 231 Tahuna Road, Ohinewai ("the site"). This assessment included a desktop review of existing geotechnical data and a site specific, preliminary geotechnical investigation. We have subsequently completed additional stages of geotechnical investigation for Lot 3, DP474347 at the western end of the site and for the proposed Stage 1 Sleepyhead development. The latter included determination of ground improvement options and the design and supervision of a field trial.

1.9 I last visited the site in August 2019 for the ground improvement trial, and for assessment of site conditions for Lot 3 DO474357 at the western end of the site, adjoining Lumsden Road.

Purpose and scope of evidence

1.10 I have been requested, in my capacity as a geotechnical engineer, to present evidence relating to:

(a) The geotechnical constraints at the site;

(b) The nature and type of ground improvement needed to mitigate geotechnical risk; and

(c) The suitability of the land for light industrial, commercial and residential purposes.

1.11 The purpose of my evidence is to address those issues. In doing so, my evidence will:

(a) Provide an overview of the local ground conditions and geology;

(b) Identify geotechnical constraints and considerations for development of the land; and

(c) Outlines how geotechnical risks and effects can be appropriately mitigated and addressed.

1.12 Specifically, my evidence will address:

(a) An overview of the proposal (Section 3);

(b) The site conditions (Section 4);

(c) Geotechnical investigations and assessment completed at the site to date (Section 5);

(d) The geology and subsurface conditions of the site (Section 6);

(e) Geotechnical constraints and considerations for future development (Section 7);

(f) Ground improvement works and other design features that are required to mitigate geotechnical risks to support future development of the land (Section 8);

(g) The geotechnical effects of the development (Section 9);

(h) A summary of further work required (Section 10);

(i) My brief conclusion (Section 11).

1.13 A summary of my evidence is contained in Section 2.

1.14 My evidence should be read together with the evidence of:

(a) Groundwater and hydrogeology covered by Mr Dave Stafford;

(b) Earthworks and sediment control covered by Mr Ben Pain;

(c) Noise and vibration covered by Mr Ben Lawrence; and

(d) Coal mining considerations covered by Mr Cameron Lines.

Expert Witness Code of Conduct

1.15 I have read the Code of Conduct for Expert Witnesses, contained in the Environment Court Consolidated Practice Note (2014) and I agree to comply with it. I can confirm that the issues addressed in this statement are within my area of expertise and that in preparing my evidence I have not omitted to consider material facts known to me that might alter or detract from the opinions expressed.

2. SUMMARY OF MY EVIDENCE

2.1 The proposed development of the site will involve earthworks, construction of civil infrastructure including roads, rail sidings, and utilities, and construction of new buildings and yards.

2.2 A series of historical and recent geotechnical investigations undertaken at the site confirm that the land is generally underlain by between 3 and 10 m of recent alluvial soils – predominantly sands and very soft to firm clays/silts and peat. Older alluvial soils comprising interbedded sands, silts, clays and peat are present beneath the recent deposits. Rock is anticipated at depths of 100 m or more below ground level.

2.3 The geology and specific ground conditions at the site present several geotechnical challenges for development. Sand layers below groundwater level are assessed as susceptible to liquefaction during seismic events. Soft soils – predominantly peat and soft clays - are highly compressible when surcharged, e.g. from new fill placed to lift the site levels and building loads. These geotechnical risks will need to be appropriately mitigated for future development on the land.

2.4 Preliminary geotechnical analyses and assessments have been undertaken to assess the relative soil compressibility due to surcharging and susceptibility to liquefaction under an ultimate limit state seismic event. With regard to soil compressibility, Figure 529-004 in **Attachment A** illustrates the estimated ground surface settlements that could occur due to surcharging of the site with an overall pressure of approximately 45 kPa. This is equivalent to approximately 2.5 m of new fill or 1 m of new fill plus loading from a typical light industrial warehouse slab. Predicted settlements range between 55 mm to greater than 2,000 mm under a 45 kPa applied pressure.

2.5 Figure 529-003 in **Attachment A** illustrates the calculated Liquefaction Severity Number (LSN) across the site for an Ultimate Limit State earthquake. The LSN is an index which was developed following the Canterbury Sequence of earthquakes and is used for categorising the effects of liquefaction for differing ground conditions and variability in soils. LSN values at the site range between 0 (no expression of liquefaction) to > 50 (severe damage).

2.6 The Sleepyhead Estate Masterplan was prepared with consideration to the key geotechnical risks at the site, particularly 'settlement' of soft soils. As can be seen from the Masterplan, proposed development has been avoided/limited in areas of the site underlain by highly compressible soil; parks and wetland reserves are proposed over the eastern and central areas of the site.

2.7 Ground improvements will be required to prepare most of the land for future development. Several different options have been considered including deep pile foundations for all buildings, stone columns or rammed aggregate piers, excavation and re-compaction/replacement, dynamic compaction and preloading. A summary of ground improvement options which could be considered at the site are presented on the table attached in **Attachment B.**

2.8 The preferred ground improvement options have been identified as dynamic compaction, excavation and re-compaction/replacement and preloading. Dynamic compaction <u>or</u> excavation and re-compaction/replacement are proposed to mitigate liquefaction severity. Preloading is recommended to mitigate post-construction settlements. In some areas of the site, ground improvements will be required to address both liquefaction and settlement risks, i.e. two types of ground improvement may be necessary.

2.9 A dynamic compaction field trial was undertaken in Allotment 405 (the north western block of land) in September 2019. Testing was undertaken prior to and following dynamic compaction to evaluate the efficacy of this method. The results demonstrated that dynamic compaction is an effective method for mitigating liquefaction susceptibility in soils extending up to 5 m below ground level. It is the preferred method for ground improvement at Ohinewai for liquefaction risk mitigation, compared with excavation and re-compaction/replacement, as it is significantly faster, less expensive and more effective; densifying soils to a depth of 5 m, compared with just 3 m for the excavation and re-compaction/replacement option. It also mitigates the risks associated with excavating below groundwater level.

2.10 Preloading will be required in most areas of the site where there is a net increase in ground surface stress due to the proposed development. Preloading involves the temporary placement of fill above final design ground level to initiate settlement in the subsurface soils to depths of up to 30 m below ground level. The preload is usually held in place until settlements are approximately 90% of the estimated long-term total or until estimated residual settlements are considered tolerable to the proposed future development. Timeframes for preloading are expected to range between 6 and 12 months depending on ground conditions, preload heights and the development type.

2.11 The geotechnical effects of the proposed development (earthworks, construction of civil infrastructure such as roads/buried services etc, and new buildings) are expected to be limited to settlement from either surcharging of ground levels – such as placement of new fill or building construction – or from lowering of the groundwater level. During construction, there may also be vibration and noise effects from Dynamic Compaction.

2.12 The offsite settlement effects of surcharging the ground from placement of new fill and/or building loads are expected to be low to negligible at distances of 10 to 20 m from the works area. Therefore, this effect can be relatively easily mitigated and controlled by avoiding or minimising surcharge close to the property boundaries where existing buildings or infrastructure are located

2.13 The effects of groundwater drawdown due to excavations below groundwater level can be controlled if necessary, by installation of 'grout curtains', sheetpile walls, 'slurry' walls or other impermeable materials/structures for cutting off groundwater flows.

2.14 Where large areas of the site are 'sealed' with impermeable surfaces such as pavements and roofs, this can have a 'rainfall shadowing' effect. A reduction in groundwater level can potentially induce consolidation of compressible soil layers such as the Rotokawau Formation peat. Where necessary, this can be mitigated by the installation of stormwater soakage devices to recharge groundwater levels. However, I note that the proposed ground improvements at the site (i.e. preloading) will effectively mitigate the effects of a reduction in groundwater level. The effects of 'rainfall shadowing' outside the site boundaries are expected to be negligible.

2.15 Dynamic Compaction field trials completed at Ohinewai have demonstrated that vibration magnitudes are expected to be less than the typically permissible magnitude of 2 mm/s Peak Particle Velocity at distances of 50 m or more from the Dynamic Compaction works. If ground improvement for liquefaction mitigation is required at distances closer than 50 m from existing dwellings, it may be necessary to employ alternative ground improvement methods such as excavation and replacement/re-compaction.

2.16 Before future development of the site proceeds, it will be essential that a comprehensive scope of geotechnical investigation and laboratory testing is undertaken to determine specific ground improvement requirements for earthworks, civil infrastructure and buildings. This would ideally be completed in stages as the development progresses. In terms of groundwater considerations and the potential need for stormwater soakage, further groundwater monitoring and assessment will be required during Resource Consent/subdivision consent stage.

3. THE PROPOSAL

3.1 The site is located on the corner of Lumsden Road and Tahuna Road, Ohinewai (Allotment 405, Lots 1 and 2 DPS 29288 and Lots 2-3 474347) on the eastern side of State Highway 1, as shown on Figure 529-001 in **Attachment A**. The proposed development, the 'Sleepyhead Estate' will be a mixed-use, master-planned community located adjacent to the Waikato Expressway and the North Island Main Trunk railway at Ohinewai.

3.2 APL has lodged a submission on the Proposed Waikato District Plan requesting that the land be rezoned to a mix of industrial, residential and business zone to accommodate the mixed-use community.

3.3 A 100,000 m² factory is proposed for The Comfort Group in Allotment 405. This will be accommodated in a 61ha industrial hub with rail siding access from the North Island Main Trunk railway.[1]

3.4 The project will also include 10ha of commercial development including a service station, local convenience stores and factory outlet shops.[2] Fifty two hectares of residential land for approximately 1100 new houses will also be provided, together with about 55 ha of public open space.

3.5 To develop the site, the land will need to be re-graded and earthworked to form design suitable building platforms, graded for stormwater and floodwater conveyance and civil infrastructure such as roads and services will need to be constructed.

4. SITE CONDITIONS

4.1 The total site area is approximately 178 ha and is presently almost completely grass covered and is used for agricultural purposes. It is bounded by Tahuna Road to the south, Balemi Road and other agricultural land to the north, Department of Conservation land, including Lake Rotokawau, to the east, and Lumsden Road to the west.

4.2 The land is typically low lying and flat except for a ridgeline on the southern boundary (the Tahuna Road ridgeline) and two "spurs" which run in a north-south direction through the two southern properties (Lots 1 and 2, DP 29288).

4.3 Ground surface elevations vary between approximately RL 20 m on the southern boundary with Tahuna Road and RL 6 m at the far eastern end of the site. Except for the localised ridge and spurs, the general site grade falls very gently from west to east. The Waikato River is located approximately 1 km to the west of the site.

5. GEOTECHNICAL INVESTIGATIONS AND ASSESSMENT

5.1 Several stages of project specific investigations have been undertaken at the site. An initial pre-purchase investigation was completed in September 2018 and included cone penetration tests, a small number of machine boreholes and test pits. A second stage of pre-purchase investigation was undertaken to evaluate the conditions of Lot 3, DP 474347 in July 2019. These two stages

1 Total area zoned Industrial is 68 hectares.
2 Total area zoned Business is 13 hectares.

of investigation were undertaken to assess the prevailing ground conditions, variability and to identify geotechnical constraints and considerations for development of the land.

5.2 A detailed stage of geotechnical investigation was completed in July and October 2019 for the proposed Stage 1 Sleepyhead development on Allotment 405. This comprised machine boreholes, cone penetration tests and geotechnical laboratory testing. The purpose of this investigation was to more comprehensively evaluate ground conditions and to identify and design the necessary ground improvements to ready the land for development.

5.3 A ground improvement trial was undertaken on Allotment 405 in October 2019 to assess the efficacy of 'dynamic compaction', one of the identified ground improvement options, for mitigating liquefaction susceptibility risk in the upper, near surface soils.

5.4 Results of the recent geotechnical investigations completed by Initia have also been supplemented by historical geotechnical investigations completed in the 1980s for a former proposed open cast coal mine. These historical investigations extended over the eastern extent of the site – in Lot 1, DPS 29288 as shown on Figure 529-001 in **Attachment A** – and included deep machine drilled boreholes extending to depths of over 100 m below ground level. An extensive suite of laboratory testing was also completed to classify the various soil types/geological units and their engineering characteristics.

5.5 A combined geotechnical investigation location plan – presenting all project specific as well as historical investigation locations – is attached as Figure 529-001 in **Attachment A**.

6. GEOLOGY AND SUBSURFACE CONDITIONS

Published geology

6.1 The published geological map is attached to my evidence in **Attachment A** (refer Figure 529-002). This shows the site is located over two surface geological formations, both members of the Tauranga Group. The Tauranga Group includes a range of marine, estuarine and terrestrial sediments deposited primarily during the Quaternary in the Bay of Plenty, Waikato and Auckland regions.

6.2 The early to middle Pleistocene Age, Tauranga Group materials are shown to be present over the elevated areas of the site, being the Tahuna Road ridgeline and the two north-south trending spurs. These soils typically comprise pumiceous river deposits comprising highly weathered, coarse pumiceous and rhyolitic sands and current-bedded grits, with interbedded peat and local gravels.

6.3 The recent, Holocene Age (2,000-10,000 years old), Tauranga Group soils are shown to be present over most of the site, away from the elevated topography close to Tahuna Road and the north-south spurs. These materials comprise pumice sand, silt and gravel with peat beds.

Subsurface conditions

6.4 The historical and recent geotechnical investigations confirm that the site is generally underlain by a surficial layer (3 to 13 m thick) of alluvial soils comprising recently deposited sands (Taupo Pumice Alluvium) and very soft to firm clays/silts and peat (Rotokawau Formation). These are Holocene Age, Tauranga Group soils. Older alluvial soils comprising interbedded sands, silts, clays and peat of the Karapiro, Puketoka and Whangamarino Formations underlie the surficial soils. These are Pleistocene Age, Tauranga Group materials.

6.5 The Tauranga Group units are underlain by basement rock (interbedded claystone, sandstone, siltstone and coal measures) known as the Te Kuiti Formation at a depth of 60 to 110 m below ground level.

6.6 Low lying areas of the site (below RL 7.5 m) are typically mantled by between 5 and 10 m of the Rotokawau Formation and Taupo Pumice alluvium. Areas of the site with higher ground surface elevations (RL 9.0 m or higher) are typically directly underlain by more competent soils (Karapiro and Puketoka Formation).

6.7 Groundwater is present from near surface levels (0.5 to 1.5 m depth) over most of the site except over the areas of higher elevation (RL > 9.0 m close to the two north-south spurs and Tahuna Road ridgeline).

7. GEOTECHNICAL CONSTRAINTS AND CONSIDERATIONS

7.1 The site is located in an area of relatively recent geology and as such there are some challenges to development from a geotechnical perspective. The principal geotechnical constraints and considerations include:

(a) Earthworks required to develop building platforms.

(b) Liquefaction susceptibility of the upper saturated, sandy soils under seismic conditions.

(c) The compressibility of the soils when subjected to surface loading (e.g. new fill, building loads etc).

(d) Installation/construction of civil infrastructure such as three water services (stormwater, sewer and water main), internal roads and yard areas, the rail siding, car parking etc.

(e) New buildings and foundations including designing for ongoing (post-construction) settlement.

Earthworks

7.2 Design surface levels are yet to be comprehensively developed; however, I expect that in general, much of the site will need to be filled and graded from west to east for stormwater and flood conveyance purposes.

7.3 Lower lying areas of the site will need to be lifted and localised elevated areas of the site (the north-south trending spurs and along the southern boundary, adjacent to Tahuna Road) will be cut down. However, the latter areas are limited and are not expected to yield enough material to achieve the design grades. I therefore anticipate that fill will need to be imported to offset the deficit of material needed to lift the site (as set out in the evidence of Ben Pain).

7.4 Cuts extending below an elevation of RL 7 in the west and below RL 6 in the east will almost certainly encounter groundwater. As the upper soils at the western end of the site are predominantly sandy, groundwater inflows into any such excavations are expected to be significant.

7.5 Excavations through the southern "ridge" and the two north-south trending "spurs" are likely to extend through a mixture of sands and silts which could be used (probably with some conditioning) for engineered fill.

7.6 Excavations which extend below the surface "crust" of upper soils (Taupo Pumice alluvium) which varies between 0.5 m and 4.5 m in thickness, will encounter very soft clay/silt or peat (Rotokawau Formation). This material is very unlikely to be suitable for re-use as engineered fill and will not support earthmoving or construction traffic.

7.7 Significant volumes of "unsuitable" soil are likely to be encountered across the site. These materials will either need to be removed off site or placed in landscaping areas away from future development.

7.8 Placement of new fill will initiate settlement of the underlying soils which may take several months to occur. The ground will also need to be over-filled to account for settlement of the deeper soils, i.e. so that design surface levels are achieved following completion of the settlement process.

Liquefaction potential

7.9 The surface soils (Taupo Pumice Alluvium) are predominantly sandy and saturated. These soils, together with the deeper Karapiro/Puketoka Formation sands, are susceptible to liquefaction under an ultimate limit state (ULS) earthquake event. There is a negligible risk of liquefaction during a SLS seismic event.

7.10 Figure 529-003 attached in **Attachment A** presents the estimated liquefaction severity number or LSN for an ultimate limit state earthquake event based on interpretation of CPT test data spread across the site. LSN values for the ULS seismic event range between 15 (minor expression of liquefaction) to up to 100 (risk of severe damage). The parts of the site directly underlain by Taupo Pumice Alluvium are most at risk of liquefaction related effects. Conversely, those areas of the site directly underlain by the Rotokawau Formation (clays and peat) have the lowest risk of liquefaction related effects.

7.11 Without mitigation measures, liquefaction of near surface soils – typically upper 2.5 to 3 m – are expected to occur during a large earthquake resulting in high magnitude settlements and potentially failure of shallow foundations which presents a 'life-safety' risk for future buildings.

7.12 Mitigation of liquefaction effects will need to be addressed for future buildings and for elements of civil infrastructure within the subdivision.

Soil compressibility and settlement

7.13 The soils at this site are considered moderately to highly compressible. The Rotokawau Formation unit in particular, is highly compressible as it is comprised of interbedded layers of PEAT and very soft clayey SILT and silty CLAY with undrained shear strengths less than 15 kPa. This unit is approximately 10 to 20 times more compressible than the underlying soils – the Puketoka and Karapiro Formations.

7.14 Settlement occurs as a result of an increase in the effective stress of the soils underlying the site. This principally occurs due to an imposed ground surface surcharge such as placement of new fill during earthworks and pavement construction, building loads (foundations and floor slab dead and live loads), and sustained live loads on pavements such as container storage. Dewatering of soils (temporary or permanent lowering of groundwater levels) can also increase the effective stress of the soils resulting in settlement.

7.15 The Rotokawau Formation soils are thickest at the eastern end of the site in Lot 1, DPS 29288 and in low lying areas between the two north-south spurs. Settlements in these areas of the site could be as much as 2,000 mm due to surface loading (1 to 2 m of new fill and building loads). Elsewhere on site, where the Rotokawau Formation soils are either absent or comparatively thin, estimated ground surface settlements from typical surface loads are expected to be significantly less – approximately 100 mm to 500 mm. Figure 529-004 in **Attachment A** illustrates the estimated total settlement of the soils across the site due to a uniform surface surcharge of 45 kPa applied which is equivalent to about 1 m of new fill plus the dead and live loads from a typical light industrial or commercial building

7.16 Settlement can occur due to initial loading (elastic settlement), consolidation of the soils over time – involving the squeezing out of porewater pressure – and secondary compression which is long-term creep movement under a constant load. Secondary compression can occur over periods of tens of years.

7.17 The settlement risk at the site will need to be mitigated over large areas of the site for future building platforms and civil infrastructure (e.g. pavements subject to sustained live loads and settlement sensitive services). Furthermore, I have recommended that development be avoided in some areas of the site where the risk of post-construction settlement is high. This is reflected in the Masterplan which shows that development is largely excluded at the most compressible areas of land through the centre and at the eastern end of the site.

Civil infrastructure

7.18 As regards civil infrastructure, I note the following:

(a) To prepare the site for light industrial, commercial, retail and possible residential development, it will be necessary to install internal roads, buried services (gravity fed services such as sanitary sewer and stormwater being of greatest relevance from a geotechnical perspective), car parking, yard areas and a rail siding off the North Island Main Trunk line.

(b) Due to the presence of sands and elevated groundwater levels, excavations for pipes and other underground services will be constrained by the stability of trenches. Therefore, the depth of buried services will be either limited to the groundwater elevation or, alternatively, pipes can be installed using temporary retention such as sheet piles. Trenchless methods such as pipe thrusting/jacking can also be used.

(c) Flexible pipes and service connections may be needed in areas that are susceptible to settlement either from liquefaction or consolidation and creep movement. Dewatering of service trenches will need to be avoided or minimised to mitigate the risk of consolidation settlement.

(d) If roads and pavements are constructed in fill areas, ground improvements may be required to mitigate post-construction settlement that could affect long-term performance.

Buildings

7.19 Because of the risk of settlement and liquefaction susceptibility of near surface soils, new buildings will need to be sited on ground improved platforms. Pile foundations can also be employed, however piling is expected to be cost-prohibitive for this site due to the great depth to a reliable bearing stratum.

8. GEOTECHNICAL DEVELOPMENT CONSIDERATIONS

8.1 Despite the geotechnical constraints outlined in the above section of my evidence, it is my opinion that the site is suitable for the proposed Sleepyhead Estate development subject to the employment of suitable ground improvements and design to mitigate geotechnical risk.

8.2 The site immediately bounds State Highway 1, the North Island Main Trunk Line – two critical infrastructure links between Auckland and the Waikato – and other factory and warehouse buildings are present in the local region. These are sited over similar geology and ground conditions.

Ground improvements

8.3 As a result of the geotechnical constraints I have summarised in my evidence, ground improvements will be required to support future development of Sleepyhead Estate site. Ground improvements are likely to be required for buildings, roads, yard areas, and the rail siding across the site.

8.4 The ground improvements are required predominantly to address the liquefaction susceptibility risk from a ULS seismic event and to mitigate the risk of settlement. In some areas of the site, more than one type of ground improvement will be required to address both settlement and liquefaction independently. In other areas, depending on the type of development, ground conditions and performance risk, ground improvements may be required only for liquefaction or only for settlement.

8.5 In summary, ground improvements are required for the following purposes:

(a) To satisfactorily mitigate liquefaction risk. Typically, I expect this will involve improvements to provide a minimum 3 m thick raft of non-liquefiable soils below any future light industrial or commercial building platform and at least 1 m below future lightly loaded residential buildings, roads and yard pavements and the rail siding. Usually, the objective of ground improvements for mitigating liquefaction risk is to ensure that life-safety, code requirements are met and achieved. The building code does not require buildings and infrastructure to be 'damage-free' following a ULS seismic event, however improvements may still be required to address economic risk to infrastructure from intermediate events (e.g. 1 in 100 year event)

(b) Minimisation of post-construction consolidation and secondary compression settlement to magnitudes that are tolerable to future buildings, floor slabs and other infrastructure subject to sustained loading (e.g. yard areas). Different building types and infrastructure will have varying degrees of tolerance to post-construction settlement. Ground improvements for settlement mitigation are usually designed and implemented to meet these threshold levels which are typically defined by the designers (civil and structural engineers). Total elimination of post-construction settlement is not required and is unlikely to be achievable at this site.

(c) Ensuring there is a subgrade of enough strength and thickness for construction of new road pavements, car parks and yard areas.

8.6 There are a wide range of ground improvement options available, varying between earthworks to preloading. Several options are considered feasible for the Ohinewai site and these are presented in the table in **Attachment C** for reference. However, the preferred options identified are as follows:

(a) Excavation and re-compaction or replacement of the upper soils to support building foundation loads and for mitigating liquefaction effects;

(b) Dynamic compaction of the upper soils for densifying soils to support building foundation loads and to mitigate liquefaction susceptibility and effects, and

(c) Preloading or surcharging to mitigate post-construction settlements to magnitudes that are considered tolerable – as advised by the relevant designers (e.g. of buildings, service etc).

8.7 I summarise the preferred ground improvement options below and outlined the type and nature of work required to implement each.

Excavation and re-compaction or replacement

8.8 Excavation and re-compaction of the upper soils - silty sand (Taupo Pumice Alluvium) to a depth of about 2.5 m to 3 m below present ground level, can be undertaken to form a ground improved raft beneath future building footprints. The sandy soils could be excavated and replaced with imported, non-liquefiable fill material such as SPR (quarry overburden), high strength clay, or GAP40 or GAP65 gravels. Alternatively, the site-won sands could be progressively re-compacted in 300-350 mm thick lifts (loose thickness) to a high, engineered standard. Lime and cement can also be blended with each layer using a hoe to improve the strength of the soil if needed.

8.9 To provide a minimum crust thickness of at least 3m below future commercial and light-industrial building slabs, new engineered fill can be imported or sourced locally to lift the site above present ground levels. New fill would ideally be SPR (quarry overburden) or alternatively cohesive soils (clays/silts) that are not susceptible to liquefaction.

8.10 Shallower improvement layers can be considered for roads and other areas of the subdivision where potential damage from a ULS seismic event does not present a life-safety risk.

Dynamic Compaction

8.11 Dynamic Compaction (DC) involves the repeated dropping of a heavy weight (generally 8-12t) from a height of about 8-12 m. This method achieves a similar, but more effective, outcome to the 'Excavation and Re-compaction/Replacement' option – i.e. it improves the upper soils to mitigate liquefaction susceptibility and effects risk - but is significantly faster and less expensive than earthworks.

8.12 The amount of energy applied to the ground - measured in tonne.metres - and the number of drops required to compact the ground adequately depends on the material type and thickness of the layer that needs to be compacted and the density needed to suitable mitigate liquefaction risk. The drop height, weight, number of drops and spacing of the drops is usually determined based on the plant available, the depth and type of material that needs to be compacted and the efficacy of the treatment.

8.13 Dynamic compaction results in ground vibrations in the proximity of the work; however, vibration magnitudes typically attenuate to acceptable magnitudes at 20 to 50 m from the drop location. Whilst most of the works are likely to be well offset from neighbouring properties, vibration effects may need to be addressed for neighbouring properties located within 50 m of the works.

8.14 Dynamic Field trials have been undertaken to confirm the viability of Dynamic Compaction and results demonstrate the feasibility and efficacy of this method for mitigating liquefaction effects. Vibration effects may need to be controlled or mitigated where work is conducted within 50 m of neighbouring properties. Please refer to the evidence of Mr Lawrence for expert evidence on ground vibrations from dynamic compaction.

Preloading

8.15 Preloading is required to address post-construction settlement associated with consolidation and secondary compression of soils extending to depths of 40 m or more below ground level. Of primary concern is the Rotokawau Formation soils (very soft silts/clays and peat) which are between 10 and 20 times more compressible that the other geological units present at the site (Puketoka Formation and Karapiro Formation). The Rotokawau Formation varies between less than 1 m and 13.5 m in thickness across the site and is thickest at the eastern end of Lot 1, DPS 29288.

8.16 Where thick layers of organic soils/peat (Rotokawau Formation) are present, e.g. at the eastern end of the site, significant long-term secondary 'creep' settlement can continue to occur for the design life of a structure and may be outside tolerable limits.

8.17 Preloading is likely to be required in the following areas:

(a) All building platforms that aren't load compensated (i.e. "unloaded" by cut earthworks) or positioned entirely over "high ground with an elevation of RL > 9 m" that is underlain directly by Karapiro/Puketoka Formation soils;

(b) All yard areas which could be subjected to medium to long term loading and the rail siding;

(c) Roads and rail siding where fill is being placed to lift the design levels.

8.18 The required preload height will depend on the nature of the development (e.g. light industrial versus commercial/retail buildings).

8.19 Settlement magnitudes and timeframes can be uncertain and ideally a trial should be undertaken to determine required preload heights, extents, timeframes and post-construction settlement magnitudes. Installation of preload monitoring instrumentation including profilometers, piezometers, extensometers and survey pins with regular monitoring is required during the preload period.

8.20 A preload period of approximately 12 months may be required to allow for 90% or most of theoretical maximum consolidation to occur from preloading. However, the preload period can be reduced by either surcharging (placing additional fill on top of the preload) or by installation of "wick drains".

Buildings

8.21 Provided that the building platforms are subject to ground improvement works to mitigate liquefaction and/or settlement risk, future buildings structures and floor slabs could be fully supported on grade, that is with shallow pad footings and ground bearing slabs.

8.22 As I have outlined above, different building types will have varying tolerance to total and differential settlements. Where there is a risk of post-construction settlement being of a magnitude that is not tolerable to the building, settlement tolerant construction methods and materials can be employed for light industrial and commercial buildings, e.g. flexible cladding systems, post-tensioned floor slabs etc.

8.23 Foundations for lightweight housing (one to two storey structures) can be designed and detailed to distribute foundation loads evenly over the footprint of the dwelling. So-called rib-raft foundation systems have been employed successfully in other areas of highly compressible ground conditions, e.g. Takanini, Auckland. In these areas, distributed foundation surcharges from two storey buildings have been limited to approximately 5 kPa over the building footprint and the structures are reportedly performing well to date. I note that settlement due to any filling of the ground to achieve higher site levels would need to be substantially complete prior to commencement of construction.

9. GEOTECHNICAL EFFECTS OF DEVELOPMENT

9.1 Geotechnical effects of the proposed development to neighbouring properties and the environment are expected to be limited. Except for earthworks and sediment and erosion control, which is addressed in Mr Pain's evidence, the geotechnical related effects of the proposed development are likely to be limited to:

(a) Possible settlement due to surcharging of the ground surface from fill placement.

(b) Potential groundwater drawdown, take and diversion of groundwater and associated settlement effects.

(c) Vibration from dynamic compaction.

9.2 I have outlined the potential effects of surcharging, groundwater drawdown and vibration from dynamic compaction to surrounding land, the environment and infrastructure below. I have also described how these effects will be mitigated, if necessary.

9.3 As I have stated earlier in my evidence, settlement can occur from an increase the effective stress of the soil. An increase in effective stress can be due to a ground surface surcharge (new fill, buildings, etc.) and also from drawing down of the static groundwater level to elevations lower than have occurred historically.

Surcharging

9.4 The offsite settlement effects of surcharging the ground from placement of new fill and/or building loads are expected to be low to negligible at distances of 10 to 20 m from the works area. Therefore, this effect can be relatively easily mitigated and controlled by avoiding or minimising surcharge close to the property boundaries where existing buildings or infrastructure are located. Detailed geotechnical analyses can be undertaken to determine the effects and minimum offsets required to mitigate this risk.

Groundwater drawdown

9.5 Settlement which occurs due to the drawing down of the static groundwater levels, by activities such as excavation below groundwater level, can affect land, buildings and infrastructure at greater offsets to surface surcharging. This is because the upper soils are predominantly sandy with high permeability, and therefore the radius of drawdown can extend substantial distances from the drawdown point/area.

9.6 Wherever possible, design surface levels of the development area will be designed to elevations above the measured static groundwater level. However, where this cannot be achieved due to other engineering considerations (e.g. grading for conveyance of stormwater), the depth and extent of groundwater drawdown will need to be analysed to determine the potential effects. If necessary, the depth and extent of groundwater drawdown can be controlled to prevent or minimise offsite effects (mostly settlement) on surrounding building, land and infrastructure. Options include installation of ‘grout curtains’, sheetpile walls, ‘slurry’ walls or other impermeable materials/structures for cutting of groundwater flows.

9.7 Drawing down of groundwater levels will also need to be controlled and avoided with service trenches that are installed below groundwater level. Seepage collars can be installed upstream of manhole risers to prevent groundwater infiltration into the stormwater system. This is a common detail for sites with elevated groundwater levels.

9.8 The potential effect of ‘sealing’ large areas of the site or so-called ‘rain-shadowing’ can reduce groundwater recharge from rainfall resulting in locally depressed groundwater levels. This has been identified as potential issue within the Rotokawau Formation geological unit, as set out in Mr Stafford’s evidence. These on-site effects can be addressed if necessary, by preloading or other ground improvement methods for mitigating consolidation settlement. Groundwater levels can also be recharged by installation of soakage pits around the site to dispose of stormwater to ground and for recharging groundwater levels. The requirement for installation of soakage pits for groundwater recharge will need to be confirmed following further groundwater/geotechnical investigation and analysis during the Resource Consent/Subdivision Consent stage.

9.9 Mr Stafford has confirmed in his evidence that there will no off-site groundwater level reduction effects in the Rotokawau Formation, because of the development, and I agree with his opinion. Further commentary on groundwater and hydrogeology effects is provided in Mr Stafford's statement of evidence.

Vibration from Dynamic Compaction

9.10 As I have outlined in my evidence, Dynamic Compaction is the preferred ground improvement option for mitigating the risk of liquefaction susceptibility and effects at this site. Dynamic compaction would typically only be completed at this site where liquefaction susceptible soils are present within 3 m of future building platforms and/or to improve the bearing/subgrade strength for foundations/floor slabs. Field trials completed at Ohinewai have demonstrated that vibration magnitudes are expected to be less than 2 mm/s Peak Particle Velocity at distances of 50 m or more from the Dynamic Compaction works.

9.11 The effects to buildings and infrastructure from vibrations with magnitudes less than 2 mm/s PPV are expected to be negligible. If ground improvement works are required at distances close than 50 m from existing dwellings, it may be necessary to employ alternative ground improvement methods such as excavation and replacement/re-compaction of the materials to a high density. I note, however, that the programme for completing bulk excavation and replacement is likely to be substantially greater than dynamic compaction.

9.12 Further commentary on vibration effects associated with the construction period are addressed in Section 11 of Mr Lawrence's statement of evidence.

10. FURTHER WORK

10.1 Additional geotechnical investigations, analysis and design will be required to confirm the specific ground improvement required for variable areas of the site and to provide detailed geotechnical advice for design of the site earthworks, civil infrastructure and individual buildings. These investigations are expected to comprise fully cored machine boreholes, groundwater monitoring installations, additional Cone Penetration Tests, test pitting and laboratory tests.

10.2 For each stage of development, it will be necessary to calculate and determine the liquefaction susceptibility and settlement risk using the results of site-specific investigations. Assessment of post-construction settlement and evaluation of the tolerance of future development to post construction settlement is also critical and will be required to assist with ground improvement design and detailing.

10.3 The potential effects to the groundwater regime will need to be considered when design surface levels are developed. Where necessary, groundwater analyses can be undertaken to determine the depth and extent of groundwater drawdown and the effects of this can then be quantified and mitigated if required.

11. CONCLUSIONS

11.1 The site is underlain by an upper layer of Holocene Age soils comprising loose sands and soft clays and peat. These soils present engineering challenges for development of the land.

11.2 The principal geotechnical engineering constraints are liquefaction susceptibility of the upper layers of loose, saturated sand (the Taupo Pumice Alluvium) during seismic events and the potential for high magnitude, post-construction settlements from surcharging of highly compressible soils (the peat and soft clays of the Rotokawau Formation).

11.3 To prepare the land for development, it will be necessary to undertake ground improvements. The objective of the ground improvements will be to:

(a) Mitigate the liquefaction susceptibility of the upper few metres of soil and to reduce the consequential effects of liquefaction such as settlement and bearing capacity failure which could otherwise present a 'life-safety' risk for future buildings.

(b) Minimise post-construction, static settlements to magnitudes that are considered tolerable to the type of development proposed on the site, e.g. housing, large footprint warehouses, roads, yard areas etc.

11.4 Several ground improvement options have been considered, however the preferred options have been identified as either dynamic compaction or excavation and re-compaction/replacement for mitigating liquefaction effects and preloading for minimising post-construction settlement effects.

In some areas of the site, more than one type of ground improvement may be required.

11.5 Where possible, development within areas of highly compressible ground conditions will be avoided. The Masterplan shows that parks and wetland reserves are proposed over the eastern and central areas of the site where the thickness of highly compressible soils is greatest.

11.6 The geotechnical effects of the proposed development on surrounding land, property and the environment are expected to be limited. The potential effects are settlement due to ground surcharging such as new fill and building loads, lowering of the groundwater level and vibrations from dynamic compaction. I have addressed each of these effects in my evidence and outlined methods for controlling and mitigating these. Consequently, it is my opinion that the geotechnical effects of the development can be adequately mitigated and controlled to no more than minor.

11.7 Despite the challenging ground conditions and geotechnical constraints outlined in my evidence, it is my opinion that the site is suitable for the proposed 'Sleepyhead Estate' development subject to the employment of suitable ground improvements and design to mitigate geotechnical risk. Further geotechnical investigation and analyses will be required to support design and consenting stages of the development.

Nicholas Ian Speight
9 July 2020

APPENDIX A

FIGURES

APPENDIX B

GROUND IMPROVEMENT OPTIONS TABLE