Our Ref.: 17029
5 May 2017

Wine Business Park Ltd
C/- Ayson & Partners Ltd
By PDF to: terry@ayson.co.nz

Attention: Terry McGrail

Dear Terry

GEOTECHNICAL ASSESSMENT REPORT
LOT 4 DP 414053, KENDRICK ROAD, RIVERLANDS

Introduction

This report presents the results of a geotechnical assessment undertaken for a proposed boundary adjustment at Lot 4 DP 414053, Kendrick Road, Riverlands. Terra Firma Engineering Ltd's (TFEL) proposal dated 5 May 2016 sets out the scope of work and conditions of engagement. Authority to proceed with the assessment was provided in writing by Steve Smith of Tangible Ltd on behalf of Wine Business Park Ltd on 6 April 2016.

The proposal consists of a boundary adjustment which results in the creation of Lot 1 (from Lot 4 DP 414053 - the Balance Lot) which will then be amalgamated with and ownership transferred to the owners of Lot 8 DP 421549. As the site was originally subdivided in 2007 and as the standard of liquefaction hazard assessments have advanced in the intervening period due to the 2010-12 Canterbury Earthquake Sequence, Council has requested that an updated[1] professional opinion be provided on the geotechnical suitability of the site for the proposed subdivision.

The general arrangement is shown on attached Figure 17029-01.

As several deep investigations have been completed in the immediate site vicinity over the last few years, no further investigations were considered necessary for the purpose of this report. It is noted however that site-specific investigations will still be required as part of any new building consent application on the site in order that due consideration can be made on the variability of the underlying soils with respect to the actual structure proposed.

This report presents the findings of the adjacent site investigations and confirms the availability of a building area together with our recommendations for development of the lot. It includes site certification in terms of Schedule 2A of NZS 4404:2010 and is suitable to accompany the application for resource consent (*subdivision*).

[1] Updated with respect to the original opinion provided by Nelson Consulting Engineers Ltd (NCE) in its report Ref 07158 dated 5 July 2007.

Geotechnical Assessment

The current geotechnical assessment comprised:

- review of previous reports and data on the site and surrounding area;
- liquefaction and settlement analyses using latest guidance & techniques;
- assessment of the geotechnical suitability of the site for the proposed industrial subdivision / boundary adjustment.

Subsurface investigations were undertaken as part of the original site certification process in June 2007 (Refer NCE report Ref 07158 dated 5 July 2007). A total of 24 digger excavated test pits (TP1 to TP24) and 11 cone penetration tests (CPT1 to CPT23 – *numbering was designed to be coincidental with the adjacent test pit*) were completed to depths of around 4m and 8m respectively. A number of shallow Scala penetrometer tests (SP2 to SP23 – *numbering as given above)* were completed adjacent to each alternate test pit location. Whilst the CPT traces were reviewed for consistency, the raw data was not available to us for further analysis. Of particular interest for this site (proposed Lot 1) are TP5 to TP8 inclusive, CPT5 and CPT8.

As part of the development of the adjacent lot at 23 Kendrick Road, a further three CPTs were completed in June 2014[2] (CPT01 to CPT03 inclusive). Two deep machine drilled boreholes (BH01 and BH02) and eight hand augered boreholes (HA01 to HA08) were also completed to depths of around 11m and 4m respectively. The raw CPT data was made available for our use.

At a second nearby site at 15 Kendrick Road, a further six CPTs and five test pits were completed by us[3] to aid with the geotechnical assessment of the property for a proposed warehouse, yet to be constructed. This data was also available for analysis.

Site Assessment

Surface Characteristics

The site (proposed Lot 1) is located at the southern edge of the Opaua River floodplain on flat-lying ground at the southwestern end of the Riverlands Industrial Park. The Wither Hills rise to the south of the site with the nearest hill being approximately 50m to 60m in height and truncated at the toe by the main trunk railway from Blenheim to Christchurch. The toe of the hill is approximately 60m to 70m from the southwestern boundary of the site.

The overall slope angle of the hill country south of the site is around 35° although it becomes steeper towards the top. Evidence of surficial failures was observed across all nearby slopes.

The subject site is bounded to the east by Lot 8 (to which it proposed to be amalgamated) and to the west by Lot 7 (DP 421549) which appears to be used for access and stormwater control. It is bounded to the south by Kendrick Road and to the north by the Riverlands Co-op Drain (which has been in this location since at least 1948[4]). The drain appears to have been designed to cater for overland flows off the subdivision. During a flood, water levels could reach around 3m depth before overtopping of the banks would occur. There was water in the drain at the time of

[2] Engeo Report Ref 11242.000.000/002 dated 17/07/2014
[3] TFEL Report Ref 1510 Dated 1/9/2016
[4] Refer Smart Maps Historic Aerial Photographs (MDC GIS System accessed 02/05/17)

our inspection (July 2016), but it was generally shallow (less than 0.5m depth) with very low flow velocity.

Sub-Surface Characteristics

The underlying geology of the site is mapped (GNS 1:250,000 Geology of the Wellington Area, Begg & Johnston, 2000) as alluvial gravels, with Hillersden Gravels being mapped beneath the Wither Hills to the south. The soils encountered during the field investigations are generally consistent with the published geology.

As given above, this assessment is based on data from three sets of investigations including CPTs, machine drilled boreholes, digger excavated test pits, hand augered boreholes and Scala penetrometer tests from 2007 to 2016 inclusive.

The available data[5] shows that in general the site is underlain by interbedded and variable layers of sands, silts, sandy silts, sandy gravels and clayey gravels. The layers vary in thickness and extent but, again in general terms only, a few distinguishable layers were determined which could be tracked across most of the site.

For the most part, a roughly 2m thick silty layer was encountered at the surface, in places overlain by colluvium (likely derived from slips off the adjacent hillside). Beneath the surficial silts lie various layers - some soft silts, some loose sands and various thin gravels. These lenses and layers of variable alluvial deposits have been built up over the underlying geological base layer – the clay bound gravels of the Hillersden Gravels that are also observed in the slopes to the south of the site. This results in a steadily increasing depth to the top of the dense gravels with increased separation from the adjacent hillside i.e. the interbedded alluvial deposits are generally thicker to the north.

Groundwater was generally encountered at around 2m depth, although this varied across the different tests and periods of testing.

Development Considerations

Recommendations and opinions in this report are based on data from Council records, published mapping and site-specific investigations completed by ourselves and others. The nature and continuity of sub-surface conditions away from the original test locations are inferred and it must be appreciated that actual conditions may vary from the assumed model.

As subsurface information has been obtained from discrete investigation locations, which by their nature only provide information about a relatively small volume of soils, there may be special conditions pertaining to this site that have not been disclosed by the investigation and that have not been taken into account in the report. If variations in the soil occur from those described or assumed to exist then the matter should be referred back to us immediately.

We consider that a building site suitable for an industrial building exists within Lot 1 (*noting this will be amalgamated with Lot 8*) as defined on Aysons Figure Ref 14161 Sheets 1 & 4 dated

27 February 2017. We consider that[6] there is not a significant risk to this building site from natural hazards provided the recommendations below are incorporated into the development.

Recommendations for Development

Health & Safety Considerations

Development of this site will involve earthworks, excavations and plant access in a busy industrial site. Appropriate care must be taken to establish safe access for vehicles and people working on the sites, as well as all normal best practice procedures for the actual construction works.

Seismic Considerations

Fault Rupture

The nearest active faults are the Vernon Fault and the Wairau Fault (a section of the Alpine Fault), which trend in an approximate east-west direction about 6km southeast and 8km northeast of the site respectively. The Tempello Fault is closer, at around 2km west of the site, but is considered to be inactive (i.e. has not moved in the last 128,000 years).

No evidence of faulting was observed during the investigations and as no fault line hazards are identified on the site per the Wairau / Awatere Resource Management Plan the risk of fault rupture hazard does not require further assessment.

Slope Stability

Although there is a relatively high and steep slope immediately to the south of the site, it is mostly offset from the site both laterally and horizontally such that in the event of failure, in either a heavy storm or seismic event, the majority of debris would be expected to back up behind the railway line and any spillage would be deposited on the road. The risk of building impact damage from deep seated instability on the adjacent hills is therefore considered to be very low.

Seismic Site Class

The New Zealand Standard NZS 1170.5:2004 lists five possible site subsoil classes. Each class gives an indication of how the site (and any overlying structures) is likely to respond during a seismic event. Classes A and B relate to strong rock and rock, while C, D and E relate to shallow soil, deep or soft soil and very soft sites respectively.

One way of differentiating between classes C, D and E is by knowing the depth to basement rock. The local basement rock was not encountered during the recent investigations. Geological cross sections of the area indicate that the depth of soil (i.e. gravels) is likely to be in the order of 250m deep. On this basis and in accordance with NZS 1170.5, Section 3.1.3, a site subsoil classification of "Class D – *Deep or soft soil sites*" may be assumed for this site.

[6] Per the wording of s106 of the Resource Management Act – incorporating the proposed changes to take effect from 1 October 2017

Liquefaction - background

Cyclic liquefaction is a phenomenon in which loose, saturated, cohesionless soils are subject to temporary but essentially full loss of shear strength due to pore pressure build-up as a result of earthquake loading.

While liquefied (and for a time afterwards) the soil is susceptible to vertical and lateral deformations in the form of flow slides, lateral spreading, ground settlements, ground oscillation and sand boils. The magnitude and mode of deformation is governed by landform, spatial continuity of the liquefiable material, soil density and the intensity and duration of cyclic loading.

The type of settlement that is most commonly estimated when liquefaction analysis is conducted is referred to as the free-field settlement. Free-field settlement is the component of land settlement that does not take account of foundation influences (e.g. loads and stiffness), or the effects of ground loss, lateral spread or strength degradation.

Liquefaction - assumptions

The assessment of liquefaction and ground deformation is not an exact process as there are a number of assumptions, variables and simplifications inherent in the empirical correlations and calculations on which the methods are based.

For this assessment it has been assumed that the potential proposed development will be an Importance Level 2 (IL2) structure as defined in Table 3.1 of NZS 1170.0. As such, the Serviceability Limit State (SLS) seismic event has a return period of 1 in 25 years. In an SLS magnitude earthquake a building designed to current New Zealand codes may suffer minor damage but should be suitable for continued use without major structural repairs.

The Ultimate Limit State (ULS) event for an IL2 structure has a return period of 1 in 500 years. A ULS earthquake is considered to be a rare, extreme event. A building designed to current New Zealand codes may be severely damaged by such an earthquake, requiring major repairs or complete replacement. The building must however remain safe for occupants until it can be evacuated.

Peak ground accelerations have been calculated in accordance with the method described in Section 5.1 of the recently released MBIE/NZGS Module 1[7]. The calculated peak ground accelerations for the site under SLS and ULS events for a Class D site are therefore 0.08g and 0.34g respectively; to be analysed with a design earthquake of magnitude 6.2 and 6.5 for SLS and ULS[8] events respectively.

Liquefaction - methodology

Site specific liquefaction analyses were undertaken on available CPT data using the computer software package CLiq v.1.7.6.49 (Geologismiki, 2006). Per the guidance in Module 1, the Boulanger & Idriss 2014 method was adopted and the liquefaction potential was calculated by

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[7] MBIE & NZGS Earthquake geotechnical engineering practice; Module 1 – Overview of the guidelines; March 2016.
[8] Module 1 requires only M=6.3 for Blenheim, but based on the recent high level of activity in the region, a higher magnitude has been adopted for the purpose of this assessment.

comparing cyclic resistance ratio with the cyclic stress ratio induced during both ULS and SLS seismic events. Fines content was calculated from the CPT results using the methods by Robertson & Wride (1998), with the method of Zhang *et al* for calculating settlements.

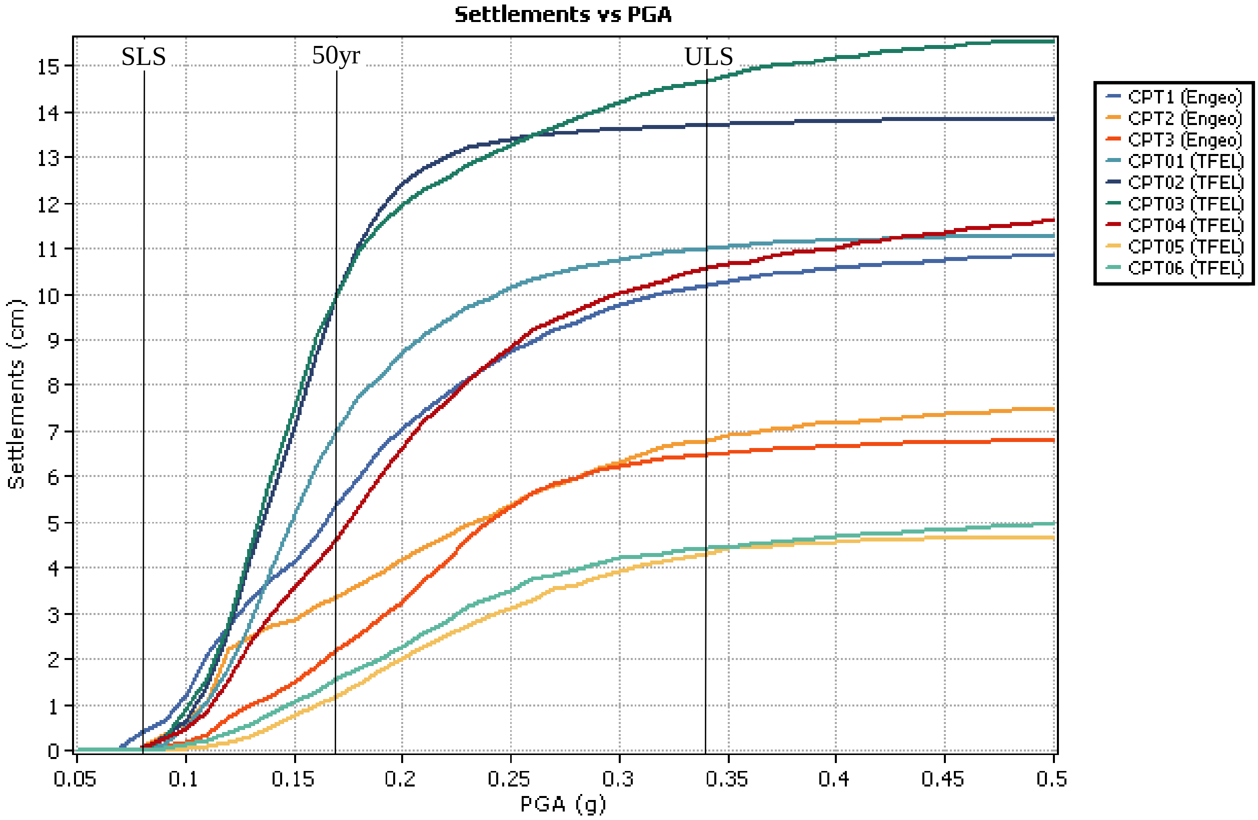
Groundwater levels used in the analysis were set just above the highest levels recorded during testing i.e. 1.5m for normal conditions and 1.2m for earthquake conditions.

The results of the analyses are shown on the attached summary plots.

Liquefaction – results

The results for the SLS level earthquake, gave very low to negligible settlements arising as a result of liquefaction (i.e. <10mm) for all CPT locations.

Figure 1 – Parametric liquefaction results showing increasing settlement for each CPT location with increasing earthquake intensity (PGA). [*Method: B&I 2014; Robertson fines; Zhang settlement*]

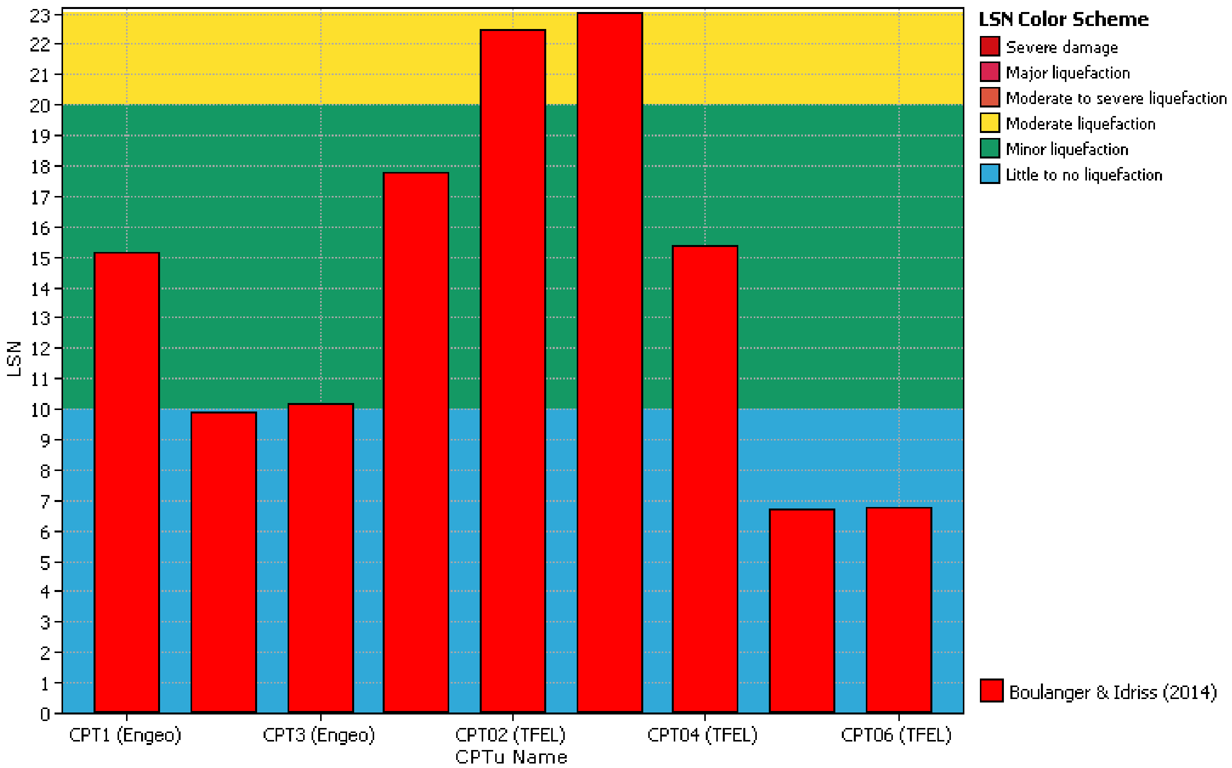


For a ULS event, the predicted settlements increase. The key result is that the settlements predicted vary considerably across the site, in particular from north to south (i.e. increasing settlements predicted with increased distance from the hills to the south). There is a generally low risk of liquefaction-induced settlement across the southern part of the site, with the thickness of liquefaction prone layers increasing and a resultant increase in settlement predicted toward the drain in the north.

Reviewing the geological conditions at NCE CPT5 and CPT8 locations and comparing them to data available at the other locations, they are most similar to conditions at Engeo CPT2 and CPT3 locations. It should be noted however, that for both the NCE and Engeo investigations, all

CPT tests were spaced out as reasonably similar distances from the hill / drain so the full effect of north-south variability may not be accurately reflected if only the results of Engeo CPT2 and CPT3 are considered.

Figure 2 – Bar chart showing variability of LSN (Liquefaction Severity Number) across tested locations. ***LSN is a measure of likely damage at the surface and takes into account the thickness and depth of each of the potentially liquefiable layers.***



In summary, total settlements in a ULS event could vary from 20mm or so in the southern part of the site up to around 150mm in the north. Differential settlements of the order of around 100mm could be expected and will need to be dealt with in the proposed development.

Although not directly relevant to this site, the MBIE Technical Categories developed to categorise various parts of Christchurch following the 2010-11 earthquake sequence can be used to allow comparison and to allow indexing of potential liquefaction issues. In terms of those categories, this site would be classified as a TC2/TC3 site.

As there is very little settlement predicted in an SLS event and considerable settlement predicted in a ULS event, it was deemed prudent to also assess liquefaction vulnerability in a 1:50 year event. This is the sort of earthquake event that could be expected to occur within the lifetime of the building (indefinite but not less than 50 years).

Depending on the method of analysis, liquefaction induced settlements could vary between 10mm or so in the southern part of the site, up to around 100mm in the north.

Although the Building Code does not specify any particular level of performance for such an event, experience in Christchurch has shown that considerable costs can arise from needing to totally rebuild a building affected by the sort of sequence of earthquakes and aftershocks associated with this type of earthquake event. Where relatively low-cost remedial options are available to provide additional resilience to the building (e.g. gravel rafts, raft foundations, stiffened foundations) these should be positively considered by the site developer in order to get the building back up and operational again in as short a period of time as possible following such an event.

Lateral Spread

Liquefaction-induced lateral spreading may occur where areas of gently sloping ground or level ground adjacent to a ‘free-face’ are underlain by laterally extensive liquefiable soils. As the underlying soils liquefy, surficial materials stretch and displace downslope. In many cases the free-face comprises a river bank or drainage channel.

At the subject site, the existing ground level is essentially flat. A drainage channel runs along the northern boundary of the site with the channel invert at approximately 3mbgl and a stormwater pond has been constructed at the eastern end of the adjacent property to the west of the site.

Considering groundwater sits at about 1.5m to 2.0m below existing ground levels, both the drainage channel running along the northern boundary and the adjacent stormwater pond introduce a lateral spread risk by creating a free face. Taking into account the increasing vulnerability of the site soils to liquefaction with increased proximity to the drain and noting that there are liquefiable layers at about the level of the drain invert, the risk of lateral spread is considered moderate to high.

Again, the incidence of lateral spread will be greater in closer proximity to the stream, with little lateral movement predicted in the southern part of the site and in the order of 2m of movement predicted in the northeastern corner of the site.

Settlement Considerations

Soft silts were encountered in most of the test pits and CPTs. The layer(s) varied up to around a metre in thickness, with the top of the layer varying from around 1.5m to 3m depth. These soft layers have the potential to compress under heavy loads from either widespread filling, widespread building loads, racking or structural loading e.g. from storage tanks. Such settlement would be independent of and in addition to any seismically induced settlement, with magnitudes varying depending on layer depth, softness, compressibility, thickness and lateral extent as well as being dependent on the load type, width and magnitude.

More detailed settlement analyses will be required once further site-specific investigations have been completed, once building plans have been developed and once all loading scenarios are known.

Recommendations for Development

Site stability

The site is considered to be at sufficient distance from the toe of the nearest steep slope, with a bund, a railway and a wide road between the base of that slope and the site. As such, although instability may occur on the adjacent slope, there is considered to be a low risk of impact damage arising as a result of such instability.

Bearing Capacity

A bearing capacity assessment will be required during the specific design stage to cater for the actual proposed structures and loads. As an indication of the density of the material '*good ground*' as defined by NZS 3604:2011 was consistently encountered in most site tests below about 0.8m to 1.0m depth. At this depth, and provided foundations extend through any fill or colluvium at that depth, a geotechnical ultimate static bearing capacity (UBC) of 300kPa may be assumed.

Foundations

Specific design of foundations by a Chartered Professional Engineer will be required once a detailed site investigation has been completed for the proposed development. The foundation design will need to take into account the likely settlements arising from both static loads and seismically-induced liquefaction, as well as taking into account the possible presence of colluvium and the likelihood of lateral spreading.

Likely suitable foundation alternatives could include:

- shallow foundations on an engineered-fill reinforced gravel raft;
- ground improvement with shallow foundations.

A piled solution is not considered appropriate where lateral spread is a possibility.

These are generic options for the proposed site and specific design will be required to cater for the actual ground conditions as well as the proposed structures and loads.

Cuts

Any proposed cuts below 1.5m to 2.0m depth during winter months will need to take groundwater into account. Dewatering may be required and as such, care taken where the excavation is located adjacent to any existing buildings, roads or other structures where any settlement associated with dewatering may occur. Specific geotechnical design input would be required in such an event.

Fills

All fill is to be placed, compacted and tested per the requirements of NZS 4431:1989. Additional, development-specific requirements may also be designated as part of the specific foundation design proposed. All fill is to be certified upon completion by a Geoprofessional[9], with all NDM test results and compaction curves appended.

Drainage

The site soil conditions are not suitable for in-ground disposal of stormwater. All stormwater is to be directed to the nearest Council-approved reticulation with appropriate treatment prior to discharge.

[9] Chartered Professional Engineer specialising in geotechnical engineering (CPEng(Geotech)) or Professional Engineering Geologist (PEngGeol), both as administered by IPENZ.

Conclusion

All proposed development on this site should require specific investigation and design inputs by a Geoprofessional. Inspections will also likely be required during construction.

A statement of professional opinion on the geotechnical suitability of land for subdivision (in accordance with NZS 4404:2010 and per Marlborough District Council geotechnical reporting requirements) is attached to this report.

Applicability

This report has been prepared solely for the use and benefit of Wine Business Park Ltd, its professional advisers, Marlborough District Council and future owners of the lot referred to herein, in relation to the specific project described. No liability is accepted in respect of its use for any other purpose or by any other person or entity. Data or opinions contained in it may not be used in other contexts, by other parties or for any other purpose without our prior review and agreement.

As subsurface information has been obtained from discrete investigation locations, which by their nature only provide information about a relatively small volume of soils, there may be special conditions pertaining to this site that have not been disclosed by the investigation and that have not been taken into account in the report. If variations in the soil occur from those described or assumed to exist then the matter should be referred back to us immediately.

Please refer any further enquiries or correspondence to Sally Hargraves.

Yours sincerely

Sally Hargraves
Principal Engineering Geologist

Reviewed by:

Andrew Palmer
Principal

Attachments: Statement of Suitability (per NZS 4404:2010)
Figures 1A & 4A (Ayson & Partners Ltd Ref 14161, dated 27/02/17)

5/05/17
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MARLBOROUGH DISTRICT COUNCIL

Statement of Professional Opinion as to the Geotechnical Suitability of Land for Subdivision

(In accordance with NZS 4404:2010 and per MDC Geo-Technical Reporting Requirements)

Issued by:	Terra Firma Engineering (2016) Ltd
To:	Wine Business Park Ltd
To be supplied to:	Marlborough District Council
In respect of:	Proposed Industrial Subdivision (Boundary Adjustment)
At:	Lot 4 DP 414053, Kendrick Road, Riverlands

I Sally Victoria Hargraves of Terra Firma Engineering Ltd of 259 Seaton Valley Road, Upper Moutere, hereby confirm that:

I am a geo-professional as defined in clause 1.2.2 of NZS 4404:2010

AND

I am an IPENZ accredited Professional Engineering Geologist (#253082).

I am familiar with and understand the purpose of the Marlborough District Council's geotechnical reporting standards. This professional opinion is furnished to the Marlborough District Council for the purpose of confirming the geotechnical suitability of the site for the proposed subdivision.

I understand that Marlborough District Council will rely on this Opinion and the accompanying Geotechnical Report for any subsequent statutory process including, but not limited to, the considerations for consent pursuant to the Building Act.

A site investigation report is attached (TFEL Ref 17029 dated 4 May 2017). A suitable number of previous site investigations have been carried out by ourselves and others and these are described in my report. I am aware of the details of the proposed plan of development as shown on the attached Figure (Ayson Ref 14161 Sheets 1 and 4 dated 27/02/2017).

In my professional opinion, not to be construed as a guarantee, I consider that the proposed works give due regard to all natural hazard considerations and that the land is geotechnically suitable for the proposed subdivision (boundary adjustment) provided that the recommendations and requirements given in my report are adhered to.

In my professional opinion, having previously examined the site and having considered any potential for geotechnical risks that may have relevance for the site, it is reasonable for Council to assume that the information referred to above is representative of the whole area under consideration (i.e. for the designated building platform and the provision of access and services to that site).

Signed for and on behalf of Terra Firma Engineering (2016) Ltd:

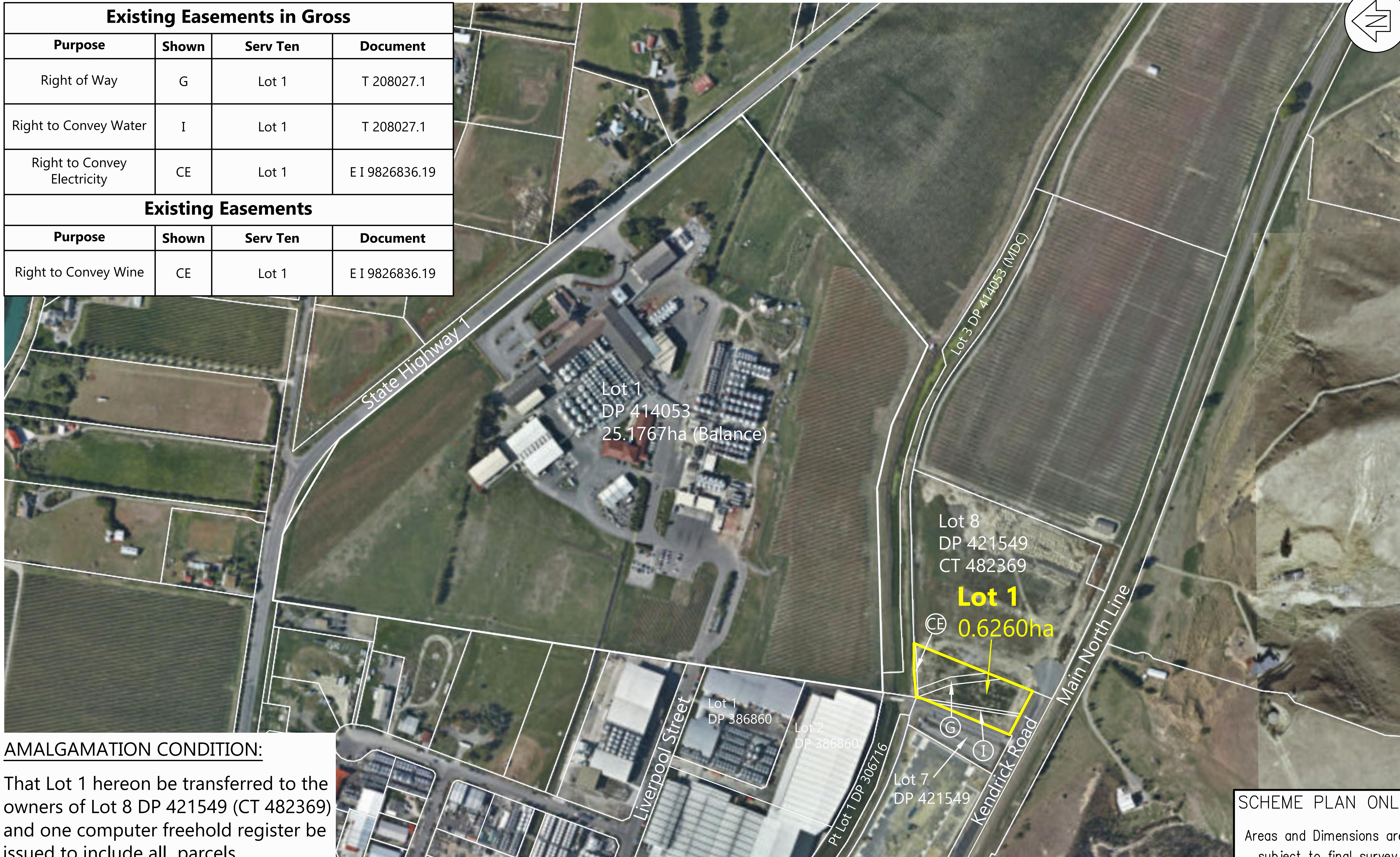
Sally Hargraves
5 May 2017
BSc PhD PEngGeol

Existing Easements in Gross

Purpose	Shown	Serv Ten	Document
Right of Way	G	Lot 1	T 208027.1
Right to Convey Water	I	Lot 1	T 208027.1
Right to Convey Electricity	CE	Lot 1	E I 9826836.19

Existing Easements

Purpose	Shown	Serv Ten	Document
Right to Convey Wine	CE	Lot 1	E I 9826836.19



AMALGAMATION CONDITION:

That Lot 1 hereon be transferred to the owners of Lot 8 DP 421549 (CT 482369) and one computer freehold register be issued to include all parcels.

SCHEME PLAN ONLY

Areas and Dimensions are subject to final survey

Ayson Survey+
RESOURCE MANAGEMENT + LAND DEVELOPMENT + SUBDIVISION

89 Middle Renwick Road
P.O. Box 704
Blenheim, New Zealand
Ph 03 579 2906
office@ayson.co.nz
www.ayson.co.nz

LOT 1 BEING PROPOSED SUBDIVISION OF LOT 4 DP 414053 KENDRICK ROAD, RIVERLANDS

COMPRISED IN : 452662

APPLICANTS : WINE BUSINESS PARK LTD

OWNERS : PERNOD RICARD WINEMAKERS NEW ZEALAND LIMITED

SCALE (A3)	JOB NUMBER	
1:4000	14161	
DATE 27.02.2017	SHEET	ISSUE
LB KB / CHECK TM	1	A

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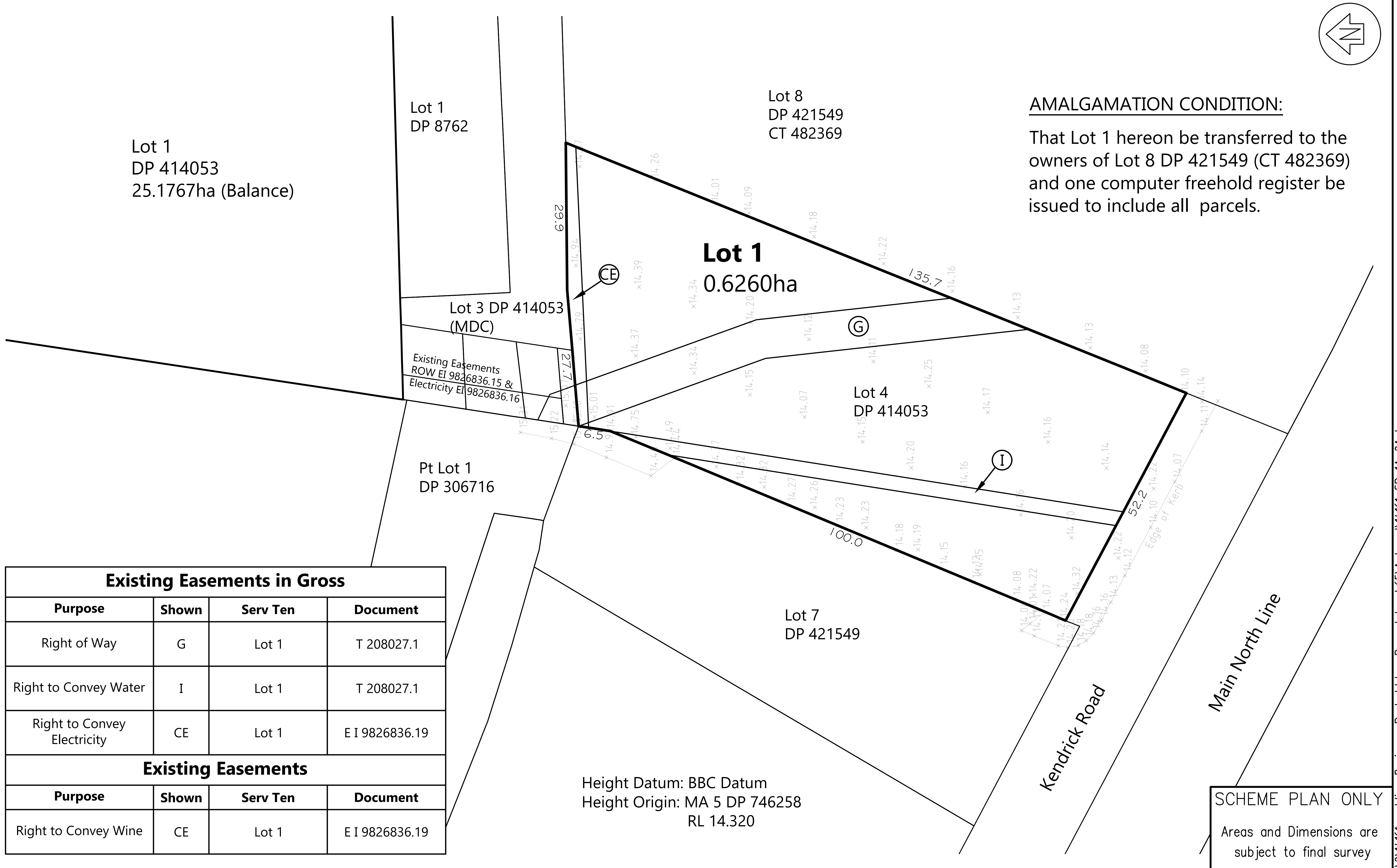

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Purpose	Shown	Serv Ten	Document
Right of Way	G	Lot 1	T 208027.1
Right to Convey Water	I	Lot 1	T 208027.1
Right to Convey Electricity	CE	Lot 1	E I 9826836.19

Existing Easements

Purpose	Shown	Serv Ten	Document
Right to Convey Wine	CE	Lot 1	E I 9826836.19

Height Datum: BBC Datum
Height Origin: MA 5 DP 746258
RL 14.320

SCHEME PLAN ONLY

Areas and Dimensions are subject to final survey

Ayson Survey+
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Blenheim, New Zealand
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www.ayson.co.nz
RESOURCE MANAGEMENT + LAND DEVELOPMENT + SUBDIVISION

LOT 1 BEING PROPOSED SUBDIVISION OF LOT 4 DP 414053
KENDRICK ROAD, RIVERLANDS
COMPRISED IN : 452662
APPLICANTS : WINE BUSINESS PARK LTD
OWNERS : PERNOD RICARD WINEMAKERS NEW ZEALAND LIMITED

SCALE (A3)	JOB NUMBER		
1:750	14161		
DATE 27.02.2017		SHEET	ISSUE
LB KB	CHECK TM	4	A

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