BEFORE INDEPENDENT HEARING COMMISSIONERS APPOINTED BY THE WAIMAKARIRI DISTRICT COUNCIL

IN THE MATTER OF	The Resource Management Act 1991 (**RMA** or **the Act**)
AND	
IN THE MATTER OF	Hearing of Submissions and Further Submissions on the Proposed Waimakariri District Plan (**PWDP** or **the Proposed Plan**)
AND	
IN THE MATTER OF	Hearing of Submissions and Further Submissions on Variations 1 and 2 to the Proposed Waimakariri District Plan
AND	
IN THE MATTER OF	Submissions and Further Submissions on the Proposed Waimakariri District Plan by **Mike Greer Homes NZ Limited**

EVIDENCE OF NEIL JAMES CHARTERS
ON BEHALF OF MIKE GREER HOMES NZ LIMITED REGARDING HEARING STREAM 12E

DATED: 5 March 2024

Presented for filing by:
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INTRODUCTION

1 My name is Neil James Charters.

2 I am a Principal Geotechnical Engineer at ENGEO. I have the following qualifications and experience relevant to the evidence I shall give:

a) I have a Master of Engineering (Dist.), Geotechnical Engineering, from the University of Canterbury, and Bachelors in Science (Hons), Engineering Geology, University of Canterbury and University of Otago.

b) I am a CPEng Chartered Professional Engineer (Number 1006195) and a member of the New Zealand Geotechnical Society.

c) I have more than 20 years' experience working with ENGEO and other geotechnical firms. My work has had a particular focus on:

i) Deep Foundations;

ii) Earth Retaining Structures;

iii) Foundation Design;

iv) Geologic Hazard Evaluation;

v) Landslide Investigations and Repairs;

vi) Liquefaction Analyses; and

vii) Slope Stability.

3 My role in relation to the Waimakariri Proposed District Plan and Variation 1 is as an independent expert witness to Mike Greer Homes NZ Limited (**Mike Greer Homes**) on geotechnical matters.

4 Although this is not an Environment Court proceeding I have read the Environment Court's Code of Conduct and agree to comply with it. My qualifications as an expert are set out above. The matters addressed in my evidence are within my area of expertise, however where I make statements on issues that are not in my area of expertise, I will state whose evidence I have relied upon. I have not omitted to consider material facts known to me that might alter or detract from the opinions expressed in my evidence.

SCOPE OF EVIDENCE

5 This evidence addresses geotechnical hazards affecting the Site and presents findings from a ground investigation and geotechnical study.

6 In preparing this evidence, I have reviewed the Geotechnical Investigation, 144 & 170 Main Road, Kaiapoi (ENGEO 24496.000.001_05/03/2024) attached as **Appendix A** (**Geotechnical Report**).

7 The Geotechnical Report addressed the geotechnical conditions relevant to rezoning of the Site. As part of this Report, ENGEO completed 15 Cone Penetration Tests (CPTs) and eight hand auger boreholes.

8 The findings of that Report are described in further detail below.

SUMMARY OF MY EVIDENCE

9 ENGEO undertook a geotechnical investigation of the site at 144 & 170 Main North Road (the Site) in November 2023. The purpose of the investigation was to inform an assessment of the Site's suitability to be rezoned from a Rural Zone to Medium Density Residential Zoning under the proposed Waimakariri District Plan (the Proposal).

10 Historically and currently, the Site has been used for primarily for agricultural purposes alongside an existing dwelling.

11 A large portion of the Site was found to be generally consistent with the medium liquefaction vulnerability category (broadly TC2 equivalent), with two areas consistent with the high liquefaction vulnerability category (broadly TC3 equivalent). These areas are shown in the appended Geotechnical Report at Appendix 1 – Figure 3.

12 A portion of the Site may be susceptible to consolidation settlement of the soft cohesive material identified in three Cone Penetration Tests (CPT) in the south central portion of the Site.

13 Based on our observations and analyses, I consider the primary geohazards to be surface deformation due to liquefaction and long-term consolidation settlement of soft cohesive or organic material across areas of the Site.

14 The risks presented by these hazards can be mitigated through earthworks and ground improvement. I therefore do not consider that these hazards should preclude the Site from being rezoned for residential purposes.

However, additional work will be required during the subdivision consent phase to refine the geological ground model and more accurately define the boundaries of these areas and the options available to remediate them.

15 Further geotechnical assessment works will be required for subdivision consent which may include: additional geotechnical testing to better characterise soil conditions, installing several piezometers to better understand the groundwater table depth and soil sampling for laboratory testing (i.e. consolidation testing and fines content).

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

16 The Site comprises approximately 14 ha of rural land across two legal lots. 144 Main North Road is legally defined as LOT 1 DP19366 BLK XV RANGIORA SD. 170 Main North Road is legally defined as PT RS 37428 RS 38486 39673 SP 17086 BLK BLK XV RANGIORA SD. The Site is currently used as agricultural grazing paddocks, with 144 Main North Road containing a dwelling and associated out buildings.

17 The Site is relatively flat and is bound by Kaikainui Stream to the north, Courtenay Stream to the south, Main North Road to the west, and an elevated railway line (Main North Line) to the east. A portion of the Site along the south-eastern boundary is lower lying, likely representing a historic river plain / channel. These features are included on the Site Plan in the Geotechnical Report at Appendix 1 – Figure 1.

18 It is proposed to rezone the Site from rural to medium density residential. A concept subdivision plan by Davie Lovell-Smith has been provided showing 200-lot residential subdivision of the Site with associated infrastructure (roading, stormwater detention basins, etc.).

DESKTOP REVIEW

Regional Geology

19 The South Island of New Zealand is located on the northeast-southwest trending boundary between the Pacific and Australian Tectonic Plates. This convergent plate boundary causes the ongoing uplift of the Southern Alps. The rapid uplift leads to high erosion rates with braided river systems supplying large volumes of eroded sediment to the coast. The Canterbury Plains are a result of these rivers depositing sediment in broad overlapping

alluvial fans. Variable sedimentation rates and changes in sea level associated with glaciation and tectonic uplift have resulted in a dynamic deposition environment producing the sequence of interbedded terrestrial, estuarine and shallow marine sediment underlying the Canterbury region.

20 The Site has been regionally mapped by GNS to be underlain by dominantly alluvial sand and silt overbank deposits of the Springston Formation, and by Forsyth et al. (2008) as being underlain by river alluvium.

21 The lower-lying portion of the Site, that extends east, is likely a historic brank of the Waimakariri River (Wotherspoon et al. 2012).

Geohazards

Seismicity

22 The nearest faults to the Site are the Loburn and Ashley faults (part of the Ashley Fault Zone), mapped approximately 16 km north and 17 km northwest respectively.

23 The Site is mapped outside of the Ashley Fault and Springbank Monocline fault awareness areas.

24 Despite being further away, the Porters Pass-Amberley Fault Zone, Hope fault, and Alpine Fault pose a significant seismic threat to North Canterbury due to their potential to produce large earthquakes.

Canterbury Earthquake Ground Shaking

25 Bradley and Hughes (2012) have developed a contour map of the conditional median peak ground accelerations (PGA) interpolated from data measured at various recording stations during the 2010-2011 Canterbury Earthquake Sequence (CES). The PGA contour map was created by combining the prediction from an empirical ground motion model of the fault rupture with the PGA recorded at any adjacent strong motion sites.

26 Based on the model by Bradley and Hughes (2012), the Site experienced the following ground motions from the two largest earthquakes of the CES:

a) A PGA of 0.20g in the magnitude 7.1 September 2010 earthquake. This is a similar level of shaking to a design 1 in 100 yr ILS earthquake.

b) A PGA of 0.19g in the M6.3 February 2011 earthquake. This is shaking similar to that of a design SLS event (1 in 25 yr).

Liquefaction

27 The Waimakariri District Natural Hazards map indicates the Site as being within an area where 'Liquefaction damage is possible' and further assessment is needed.

28 Aerial photographs of the Site taken after the September 2010 earthquake show large quantities of liquefaction ejecta within the southeast lower lying portion of the Site, but no obvious signs liquefaction ejecta elsewhere across the Site.

Flooding

29 We have reviewed the Waimakariri District Council GIS database and the mapping indicates that parts of the eastern and southern sides of the Site may be subject to a high flood hazard (defined as inundation of extremely high depth and / or water velocity) during a 1 in 200 year flood event. A medium flood hazard (defined as inundation of depth greater than 0.3 m) has been associated with the lower-lying portions of the Site extending along the western and eastern sides of the Site for the 1 in 200 year flood event. Flood hazard assessment is outside of our scope of work, and I understand this is being addressed by others to support the plan change submission.

Tsunami

30 Assessing the risk from tsunami is outside of our scope of work but we have reviewed the Waimakariri District Council GIS database in relation to the tsunami evacuation zones. The Site is located within the Orange evacuation zone, which is an area which could be impacted by a 500 year return period tsunami. It includes low-lying coastal areas that are likely to be flooded in a large tsunami that inundates land.

Historical Aerial Photography Review

31 We have reviewed historic aerial photographs of the Site available through Canterbury Maps (Property Search) dating back to 1940 as outlined below:

32 The Site appears to have been used as agricultural grazing land since the earliest available photograph in 1940. The dwelling at 144 Main North Road and the railway line was developed prior to the 1955 to 1959 aerial photograph.

33 Potential paleochannels (remnants of inactive stream channels or water flow paths) are noted on the western and eastern sides of the Site in the 1960-1964 aerial photograph. These are generally orientated northeast to southwest and align with the medium flood level hazard areas.

34 Aerial photographs following the September 2010 earthquake event show moderate to severe liquefaction ejecta and extensive lateral spread ground cracks in the lower lying portion of the Site that extends east of the Site (i.e. the historic branch of the Waimakariri River). In this area large ground cracks are visible running approximately parallel to the Courtenay River and indicate lateral spreading in this direction.

35 Aerial photographs following the February 2011 earthquake event show minor to moderate liquefaction ejecta and associated ground cracks on the other side of the railway corridor to the east of the Site. No clear liquefaction or ground cracks are visible on the Site itself.

36 The Kaikainui and Courtenay Stream channels appear to be relatively consistent with current day across the reviewed historic aerial photographs.

SITE INVESTIGATION

Site Observations

37 ENGEO visited the Site on 18 January 2024 and made the following observations:

a) The vast majority of the Site area is grassed paddocks used for agricultural grazing.

b) A residential dwelling was present on 144 Main North Rd with an associated garage and small sleepout building. A garage and barn were present to the east of the dwelling in the paddock area. The remainder of the Site was being used for agricultural grazing (sheep).

c) The Kaikainui Stream present along the northern boundary line was flowing during the Site visit. The banks of this stream are relatively steep and vegetated on either side of the banks. The bank height of the stream is approximately 2.0 m.

d) An approximately 1 m deep ditch / drain was observed along the northern boundary line of
144 Main North Road which then presumably went to an underground drain

then into a small tributary stream and discharged to the south at Courtenay Stream. There was only a small amount of stagnant water observed in the drain.

e) Courtenay Stream present along the southern boundary line was flowing at the time of the Site. The stream is relatively wide and has areas of flat sections of bank on the northern side (on the Site), and a bank height of approximately 2.5 m to 3 m.

f) A railway is present along the eastern / south-eastern boundary line of the Site. The railway is built up above the Site (approximately 1.5 m to 2.0 m high bank) and runs along the entirety of the Site boundary.

Subsurface Investigations

38 A site-specific geotechnical investigation programme, including eight hand auger boreholes and fifteen Cone Penetration Tests (CPTs), was undertaken by ENGEO in November 2023.

39 The density of testing was guided by Table 2.1 of the MBIE (2021) Earthquake Geotechnical Engineering Practice, Module 2 (Recommended Minimum Deep Geotechnical Investigation Intensity for Plan Change or Subdivision Consent Applications).

40 Investigation locations are shown in the Geotechnical Report at Appendix 1 - Figure 2. Full logs are presented in Appendix 2 of the Geotechnical Report and are written in accordance with the New Zealand Geotechnical Society field classification guidelines (NZGS, 2005). Interpreted findings from these investigations are discussed in Paragraphs 42 to 47

ENGINEERING GEOLOGICAL MODEL

Subsurface Profile

41 The subsurface testing indicates highly variable subsurface conditions across the Site. The lower lying portion of the Site represents a historic branch of the Waimakariri River and the variability in ground conditions is consistent with the alluvial depositional environment where rivers have avulsed across the landscape over time, creating a layered subsurface profile comprising clay, silt, sand, gravel, and organic deposits.

42 We have developed an engineering geological model (EGM) using our understanding of the geology and geomorphology of the Site and the results

of on-site investigations. Our ground model is illustrated by a number of geologic cross sections through the Site included in Appendix 3 of the Geotechnical Report. A generalised discussion of ground conditions across the Site is provided below and a summary of soil units is provided in Table 2 of the Geotechnical Report.

a) The upper 2.5 m of the soil profile is somewhat consistent across the Site and includes interbedded loose sand-silt mixtures (Unit 2) and clay-silt mixtures (Unit 3). Below this depth ground conditions are complex and highly variable but can be broadly categorised into three ground conditions zones. The locations of these zones across the Site are shown in the Geotechnical Report at Appendix 1- Figure 2 and described below.

   i) Zone 1 (shallow gravels - northern end of Site - CPT01-06) - Sand-gravel mixtures (Unit 6) extend from the base of the upper deposits to approximately 12.5 m depth.

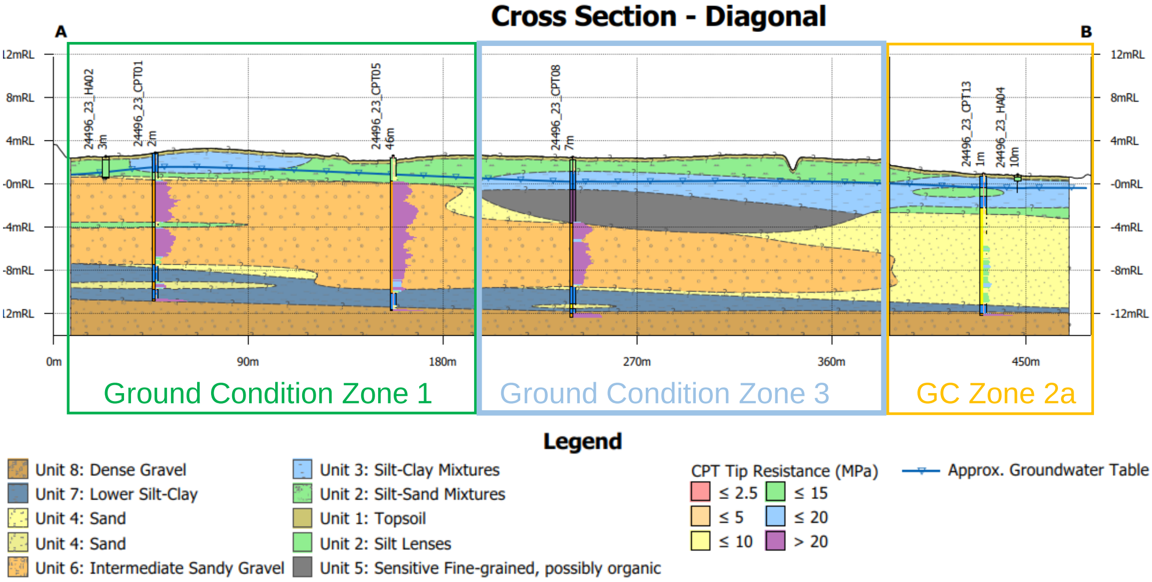
   ii) Zone 2 (sand-silt and sand – eastern and western sides of the Site - CPT07, 09, 10, 12, 13 ,15) - Sand and sand-silt mixtures extend to 12 m depth. The portion of Zone 2 (Zone 2a) along the east side of the Site, comprises the lower lying area that represents a historic branch of the Waimakariri River.

   iii) Zone 3 (soft soils – southern central portion of Site – CPT08, 11, 14) – Clay-silt mixtures (Unit 3) overlie soft, sensitive, fine-grained soils (possibly peat and / or organic silts, Unit 5) from 4 m. The depth of the Unit 5 increases from 6 m depth to 10 m depth toward the south. This is underlain by sand-gravel (Unit 6) to approximately 13 m depth.

b) From around 12 to 13 m depth, a firm to stiff silt-clay (Unit 7) with occasional lenses of sand extends across the Site. Below this, CPT investigations refused on an inferred dense gravel layer (Unit 8) encountered typically between 13 m and 14.8 m.

43 The cross sections included in Appendix 3 of the Geotechnical Report provide a more comprehensive understanding of the distribution of the various soil units across the Site than the generalised discussion above. A diagonal cross section through the Site captures each of the three zones and is reproduced in Figure 1.

Figure 1: Diagonal Cross Section with Inferred Zones



Groundwater

44 The depth to groundwater typically varied across the Site between 1.0 m and 2.5 m depth. Shallower groundwater was typically present in the lower lying area of the Site to the southeast and along the northwest Site boundary. Deeper groundwater was encountered in the southwest portion of the Site.

45 Dissipation testing was carried out at the majority of CPT locations within the lower dense gravel layer. However, groundwater levels indicated by the pore pressure readings typically indicated a groundwater table metres lower than the measured groundwater depth from dipping the CPT and hand auger holes. This indicates that the bottom gravel layer is hydraulically disconnected from the upper aquifer by the overlying clayey layers.

46 At this early stage of geotechnical assessment, a design groundwater table of 1.0 m below ground level (bgl) has been adopted for preliminary liquefaction triggering analysis. This should be refined during subdivision consent stage of works.

Ground Model Assumptions and Uncertainties

47 No visual description has been carried out for the soils below the maximum depth of the hand augers. The ground conditions below this have been inferred from the CPT soil behaviour types only. It is recommended that the next phase of ground investigation includes machine boreholes across the Site to allow logging of the deeper soils.

48 CPT's provide limited understanding soft soils and compressibility potential, as there is no sample recovery with this method. We have therefore had to infer between the silt-clay mixtures and the potentially organic material.

49 The high variability in the ground conditions, and relatively low density of investigations means there is a great deal of uncertainty in ground conditions particularly at the boundaries between ground conditions zones. We have made rough approximations of the extent of each zone and the continuity of layers. Additional investigations and refinement of the ground model will be required at further development stages.

50 Additional investigation of groundwater levels will be required to better understand the groundwater regime. I recommend wire piezometers are installed during subsequent ground investigation phases to allow continuous monitoring of groundwater levels at multiple locations across the Site.

GEOTECHNICAL ASSESSMENT

51 Based on our review of mapped land damage following the September 2010 earthquake and the ground conditions encountered in our investigations, we consider surface deformation due to liquefaction and consolidation settlement of the soft compressible soils to be the primary geotechnical considerations for the planned development. We have carried out assessments of the liquefaction potential and a preliminary assessment of the long-term consolidation settlement potential of the Site using on-site CPT data. Results and analysis are outlined in the following sections.

Seismic subsoil class

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

Liquefaction Assessment

53 Soils potentially susceptible to liquefaction were encountered in on-site investigations, particularly in Ground Conditions Zone 2 where relatively thick deposits of loose to medium dense sands and silts (Units 2 and 4) were encountered.

54 Following the September 2010 earthquake, the historic branch of the Waimakariri River lying north of Courtenay Stream suffered extensive lateral spread and liquefaction damage, particularly along the north edge of the

historic channel boundary (Wotherspoon, 2012). This lower lying area extends into the Site along the southeast edge (Zone 2a). Aerial photos taken following the September 2011 earthquake show extensive lateral spread cracks and liquefaction ejecta within this area along the southeast side of the Site.

55 We have undertaken a liquefaction assessment using the on-site CPT data for various earthquake scenarios (25yr, 100yr, 150yr, 500yr Return Period) as discussed in the Geotechnical Report.

56 Liquefaction vulnerability varied across the Site with the highest liquefaction vulnerability in Ground Condition Zone 2 where the thickest deposits of loose to medium dense silts and sands were encountered. Details of our liquefaction analysis and full results are provided in Appendix 4 of the Geotechnical Report and liquefaction vulnerability maps are included in Figures 3 and 4 in Appendix 1 of that Report. .

57 Liquefaction vulnerability and confidence in our assessment broadly falls within three areas as described below:

a) Zones 1 & 3- Medium Liquefaction Vulnerability, Reasonable Confidence: These were the locations where shallow dense gravel was encountered or cohesive deposits dominated the upper soil profile. There is reasonable confidence in the vulnerability across this area as analysis results are broadly consistent with Site performance in the September 2010 earthquake (no obvious signs of liquefaction), and the lack of significant liquefiable layers present. There is greater uncertainty where this area borders other areas as the investigation density is relatively low.

b) Zone 2a – High Liquefaction Vulnerability, High Confidence (CPT09,12,13): This area forms the lower-lying southeast portion of the Site. Extensive liquefaction and lateral spreading damage is expected within this area in ILS and ULS events based on both Site performance in the CES and our analysis. As such there is high confidence in our categorisation of high liquefaction vulnerability.

c) Zone 2b – High Liquefaction Vulnerability, High Uncertainty (CPT07, 10, 15): In this area analysis results indicate high liquefaction vulnerability, but no obvious signs of liquefaction were observed in September 2010 indicating that our analysis may be overpredicting liquefaction vulnerability. As such there is higher uncertainty in these results. One potential cause for this discrepancy across a portion of the Site is the assumption of a 1 m deep groundwater table. Investigations on the south end of this Zone indicated a groundwater depth on the order of 2 to 2.5 m, more consistent with the water levels in the adjacent Courtenay Stream. The adoption of a deeper groundwater table (say 2-2.5 m) would impact the liquefaction results at this location, but additional groundwater investigation is required to support the adoption of a deeper groundwater table for analysis.

58 As the boundaries of these zones are based on a relatively low density of investigations the uncertainty in category becomes high near the boundary with other categories. Further investigation at subdivision consenting stage is required to delineate and refine these boundaries.

Lateral Spread

59 We have undertaken a preliminary assessment of lateral spread potential toward Kaikainui and Courtenay Stream under an ILS event. It should be noted that lateral spreading mechanisms are complex, are often not adequately captured by the available simplified procedures, and at this Site are based on limited data. We emphasise that our analysis is very preliminary, and there is high uncertainty in the indicated offset zones for Zone 1,2b, and 3. More investigation and assessment of lateral spreading is required at subdivision consent stage.

60 We have run an initial analysis using the methods of Zhang (2004) and Youd (2002). It is our experience that results of lateral spreading analyses using the methods above tend to be conservative when compared with actual Site performance, and results are highly sensitive to the depth of the groundwater table. As such we have used "best estimate" groundwater depths. An initial indication of lateral spread potential is provided for each zone below.

a) Zones 1 and 3: We anticipate lateral spreading deformations toward Kaikainui Stream and Courtenay Stream may be significant (up to 250 mm of lateral displacement), within around 20 m from the stream channel in an ILS event. These offsets from the stream banks are shown in Figures 3 and 4 in Appendix 1 of the Geotechnical Report. We emphasise that additional analysis is required and additional investigations are needed to refine this estimate.

b) Zone 2a: We anticipate extensive lateral spread damage in a future ILS event across this area based on the performance of this area in the September 2010 earthquake.

c) Zone 2b: As a preliminary estimate lateral spreading toward Courtenay Stream may be significant within around 20 m from the stream in an ILS event (similar to Zones 1 and 3), but there is high uncertainty in this estimate as it is based on very limited data. Lateral spread potential is sensitive to the depth of the groundwater table. We have used a "best-estimate" depth of 2.5 m along this portion of Courtenay Stream but additional investigations are required and if shallower groundwater is found, the zone of significant lateral spreading will likely be greater than 20 m.

61 As discussed earlier in this report, the east side of the Site gently slopes toward a lower lying area where extensive liquefaction is expected. Aerial photographs taken following the September 2010 earthquake did not show obvious signs of lateral spreading of the ground at the top of this slope toward the lower lying area, and we consider that the risk of lateral spreading at the top of this this slope is relatively low. However, if the lower lying area is to be further excavated or the slope is to be steepened / heightened during development earthworks, the risk of lateral spreading may increase and this should be assessed.

62 We anticipate that bulk earthworks may form areas of sloped ground such as for stormwater basins. These areas of sloped ground can be subject to lateral spreading and the development design will need to assess and appropriately manage the risk of lateral spread. This could include creation of development offset zones from the slope and / or the need to install retaining walls or inground walls depending on slope steepness and the proximity of buildings and roads. Any services crossing slopes at risk of lateral spread will need to be designed accordingly. Additional lateral spread analysis will be required once the civil/landscaping design of the site-wide landform has been developed.

Consolidation Settlement

63 Weak cohesive and possibly organic soils identified in the CPTs within Zone 3 can be susceptible to consolidation settlement over time under new loading from earthworks fill or structure loads. As an initial screening of consolidation potential on this Site, we have completed a preliminary settlement calculation

using the CPT data over a range of loading representing load from fill placed across the Site.

64 The analysis indicates the soft soils found within the Zone 3 area of the Site are susceptible to consolidation settlement and settlement may be significant (greater than 50mm) even under relatively moderate fill loadings (10 kPa – 0.5 m of fill) in this area. We recommend additional investigation and assessment be undertaken to further define the consolidation risk once fill loads, particularly within Zone 3, are understood. Preliminary settlement estimates in Zones 1 and 2 were generally minor (less than 20 mm) under 20 kPa loading, and these zones are considered to have low risk of consolidation settlement.

SUMMARY OF FINDINGS AND RECOMMENDATIONS

65 Based on our findings and analyses outlined above, we consider the primary geohazards to be surface deformation due to liquefaction and long-term consolidation settlement of soft cohesive or organic material across areas of the Site (see Geotechnical Report at Appendix 1 for ground conditions zones and hazard maps).

66 The risks presented by these hazards can be mitigated through earthworks and ground improvement, as outlined in Table 1. In some areas, these works may be extensive, but are relatively standard methods used routinely in the Canterbury Region.

Table 1: Preliminary Geohazard Implications / Mitigation Approaches

Geohazard		Applicable Zone	Potential Mitigation / Implications
Liquefaction	Medium Vulnerability	Zone 1 & 3	Structures within these areas may require shallow ground improvement such as a gravel raft, or adoption of rigid foundations capable of tolerating some liquefaction-induced deformations. Gravity services may require greater minimum falls to accommodate anticipated liquefaction settlements.
	High Vulnerability	Zone 2	Development within these areas likely require deep ground improvement to increase the liquefaction resistance of the soils. Appropriate ground improvement depends on the particular site conditions but may include stone columns or vibratory compaction.
Lateral Spreading		Areas adjacent to the streams or sloping ground, in both medium and high liquefaction vulnerability zones	Options for reducing lateral spread risk include: Improving the ground adjacent to the sloping ground or free face (i.e. stream bank). This is typically achieved by an in-ground palisade wall, or an array of deep ground improvement columns (e.g. soil mixed column or stone columns). Defining a building exclusion zone along the sloping ground or free face beyond which lateral deformations are tolerable.
Consolidation Settlement		Zone 3	For portions of the Site at risk of damaging consolidation settlement, preloading is likely an appropriate option. Preloading involves adding load (usually soil fill) and allowing settlement to occur before removing the load and / or constructing foundations.

67 In some areas, mitigation works may be extensive, but are relatively standard methods used routinely in the Canterbury Region. We therefore do not

consider that these hazards should preclude this Site from being rezoned for residential purposes under the proposed Waimakariri District Plan.

68 However additional work will be required during the subdivision consent phase to inform subdivision design and associated bulk earthworks and to support any residential development. This additional work will allow for refinement of the geological ground model to more accurately define the boundaries of hazard areas and further define the options available to remediate them. I anticipate these works may include:

a) Additional geotechnical testing across the Site to further delineate the boundaries between the ground condition zones, and liquefaction vulnerability categories outlined in the Geotechnical Report. These investigations should include machine boreholes to ground truth the areas with potentially compressible soils and to identify the extent of the gravel deposits encountered in the investigations to date.

b) Additional lateral spreading analysis will be required once additional investigations have been carried out and the development / earthworks plan is known. This can be completed during the subdivision consent stage of development.

c) Further assessment of consolidation settlement will need to be completed within Zone 3 during subdivision consent stage. The density and extent of testing is dependent on the earthworks and development proposed. However, testing will likely consist of machine boreholes, and consolidation laboratory testing.

d) Groundwater monitoring, especially in Zone 2b where this could have a significant impact on the liquefaction potential of this portion of the Site. It is recommended that standpipe and / or vibrating wire piezometers are installed during subsequent ground investigation phases to allow continuous monitoring of groundwater levels across the Site.

CONCLUSION

69 Overall, I consider that there are no geotechnical issues or hazards with this Site which would preclude it from being rezoned for residential purposes, as sought by the landowners in their submission on the proposed Waimakariri District Plan. While we have identified a number of geotechnical

issues/hazards with the Site, I consider that these can be appropriately addressed at the subdivision stage, with the benefit of additional geotechnical assessment works (which can be undertaken once a subdivision design has progressed).

70 Thank you for the opportunity to present my evidence.

Neil Charters
Date: 5 March 2024

APPENDIX A

Geotechnical Investigation, 144 & 170 Main Road, Kaiapoi (ENGEO 24496.000.001_23/2/2024)