



ANZ Centre, 267 High Street,
PO Box 13960, Christchurch,
8141, New Zealand
T: +64 3 366 3521 // F: +64 3 366 3188
E: info@beca.com // www.beca.com

Ōhoka Residential Subdivision Expert Panel Chair

7 October 2026

Attention: Kate Barry-Piceno

Dear Kate

Ōhoka FTAA – Geotechnical Review

Introduction

Purpose of the document

1. This document summarises my review of the geotechnical assessment for the proposed Ōhoka residential development. The review excludes consideration of site contamination and the development's effects on groundwater.

Documents reviewed

2. I have referred to the following documents as part of my review:
 - Novo Group, Fast-track Approval Application, Resource Consent Applications, Wildlife Act Permit and Archaeological Authority for Ohoka Residential Subdivision. 19 June 2026. RC255196 FTAA - 2606 - 1253 OHOKA RESIDENTIAL SUBDIVISION AEE CARTER GROUP WAIMAKARIRI LIMITED 531 MILL ROAD OHOKA.PDF
 - Tetra Tech Coffey, Ohoka Development, Geotechnical Assessment Report – Fast track Application, rev R0, 16 June 2025. RC255196 FTAA - 2606 - 1253 GEOTECHNICAL ASSESSMENT CARTER GROUP WAIMAKARIRI LIMITED 531 MILL ROAD OHOKA.PDF
 - PDP, Fast Track Application – 531 and 535 Mill Road, Ohoka – Dewatering and Dust Suppression Assessment of Environmental Effects, 7 May 2026. RC255196 FTAA - 2606 - 1253 DEWATERING & DUST SUPPRESSION ASSESSMENT CARTER GROUP WAIMAKARIRI LIMITED 531 MILL ROAD OHOKA.PDF
 - Civil Engineering Plans – Inovo, Mill Road Ohoka Waimakariri Canterbury Subdivision Development, For Fast Track Consent Application, Project #16013, Date June 2026 - RC255196 FTAA - 2606 - 1253 CIVIL ENGINEERING PLANS CARTER GROUP WAIMAKARIRI LIMITED 531 MILL ROAD OHOKA.PDF
 - Groundwater Assessment (Hydrology Assessment) – PDP, Fast Track Application - 531 and 535 Mill Road, Ohoka – Hydrology Assessment, Revision 5, Dated 02/06/2026- RC255196 FTAA - 2606 - 1253 HYDROLOGY ASSESSMENT CARTER GROUP WAIMAKARIRI LIMITED 531 MILL ROAD OHOKA.PDF

Key areas discussed in this letter

3. Key areas discussed in this letter are:
 - a. Design standards



- b. Depth of geotechnical investigation completed
- c. High groundwater
- d. Liquefaction triggering potential
- e. Proposed conditions and review of detailed design at engineering approval stage.

Qualifications

- 4. I hold a Bachelor of Engineering with Honours (Resource) from the University of Auckland and am a Chartered Professional Engineer (Geotechnical) [CPEng] in New Zealand and a Registered Professional Engineer of Queensland (RPEQ).
- 5. I am a Chartered Member of Engineering New Zealand, a member of the New Zealand Geotechnical Society (NZGS) and a member of New Zealand Society on Large Dams (NZSOLD).
- 6. I have 25 years' geotechnical engineering experience, with a focus on geotechnical assessment and design, earthquake geotechnical engineering, horizontal infrastructure design and resilience assessment.
- 7. My experience in geotechnical design and risk management is from a wide range of projects and includes assessment of land for subdivision, liquefaction assessment, stability assessment, foundation design for buildings and civil infrastructure (i.e bridges, pump stations, substations, wharfs), pipeline assessment, dam assessment and design of flood defenses and retaining structures. I have also independently reviewed geotechnical designs for residential and commercial developments, roads and motorways. I have verified a range of multi hazard study's in the South Island in Christchurch, Glenorchy and the Southern Corridor in Queenstown. Following the 2010/2011 earthquakes I was involved with the rebuild in Canterbury, spending 4 years within the Stronger Christchurch Rebuild Team (SCIRT) leading a geotechnical design team. I have also previously assisted Waimakariri District Council with geotechnical engineering peer review support for Stage 3 of the Revenswood subdivision in Woodend.
- 8. I contributed to the Ministry of Business, Innovation and Employment (MBIE) guidance document *Planning and Engineering Guidance for Potentially Liquefaction-Prone Land* (2017). I also led the development of the Guideline for Assessing the Technical Resilience of Three Waters Networks (Quake Centre, 2019).

Code of Conduct

- 9. I confirm that I have read and agree to comply with the Code of Conduct for Expert Witnesses contained in the Environment Court Practice Note 2023. This technical review has been prepared in accordance with that Code. In particular, unless I state otherwise, the opinions I express are within my area of expertise, and I have not omitted to consider material facts that might alter or detract from the opinions that I express.

Key Issues

- 10. The site, and land adjoining the site, exhibited evidence of earthquake-induced excess pore pressure effects during the 4 September 2010 and 22 February 2011 Canterbury earthquakes. Liquefaction mapping interpretation from aerial photography and satellite imagery obtained from Canterbury Maps (sourced from Environment Canterbury's Natural Hazards Research Platform Liquefaction Study) indicates that the site and surrounding area were affected by strong ground shaking. Evidence included mapped liquefaction ejecta, localised surface flooding with sediment, and areas of surface flooding. The flooding may be associated with the dissipation of excess pore pressures within the underlying soils, influenced by shallow groundwater and/or springs.

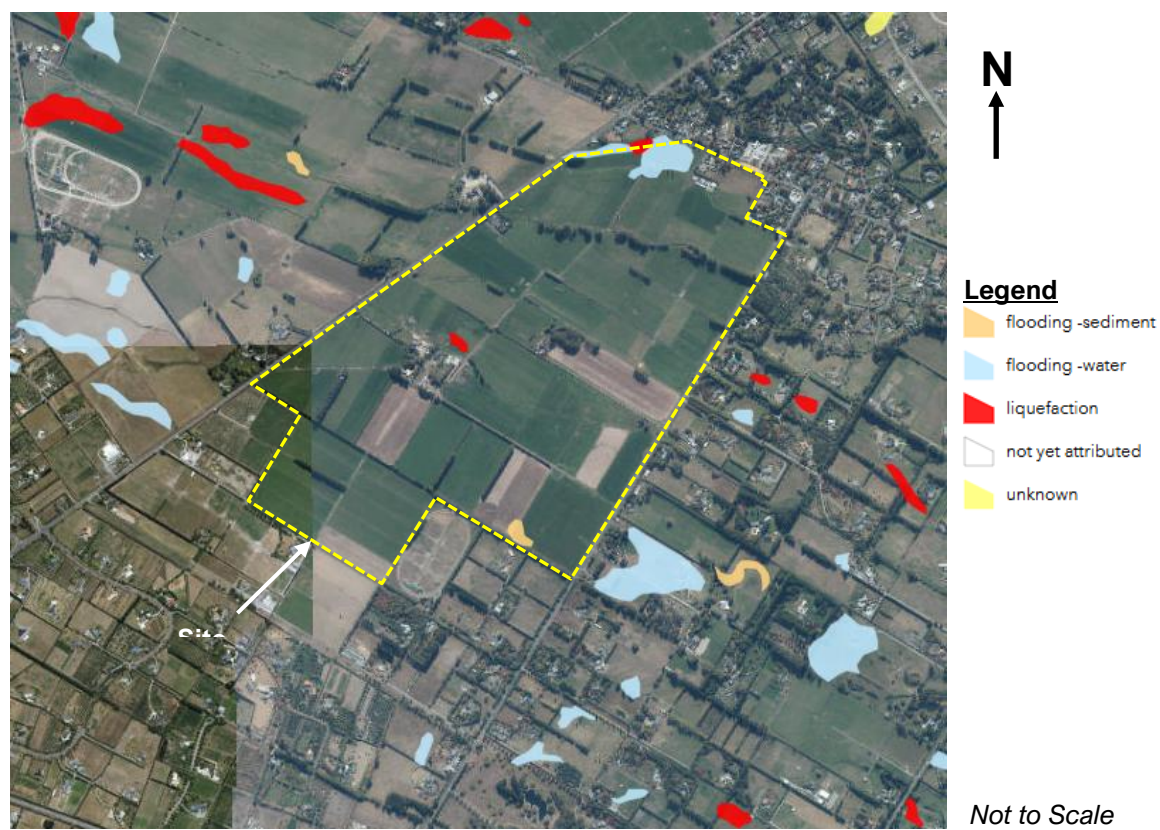


Figure 1: Environment Canterbury aerial and satellite imagery observation of inferred liquefaction following 4 September 2010 (data source Canterbury Maps)

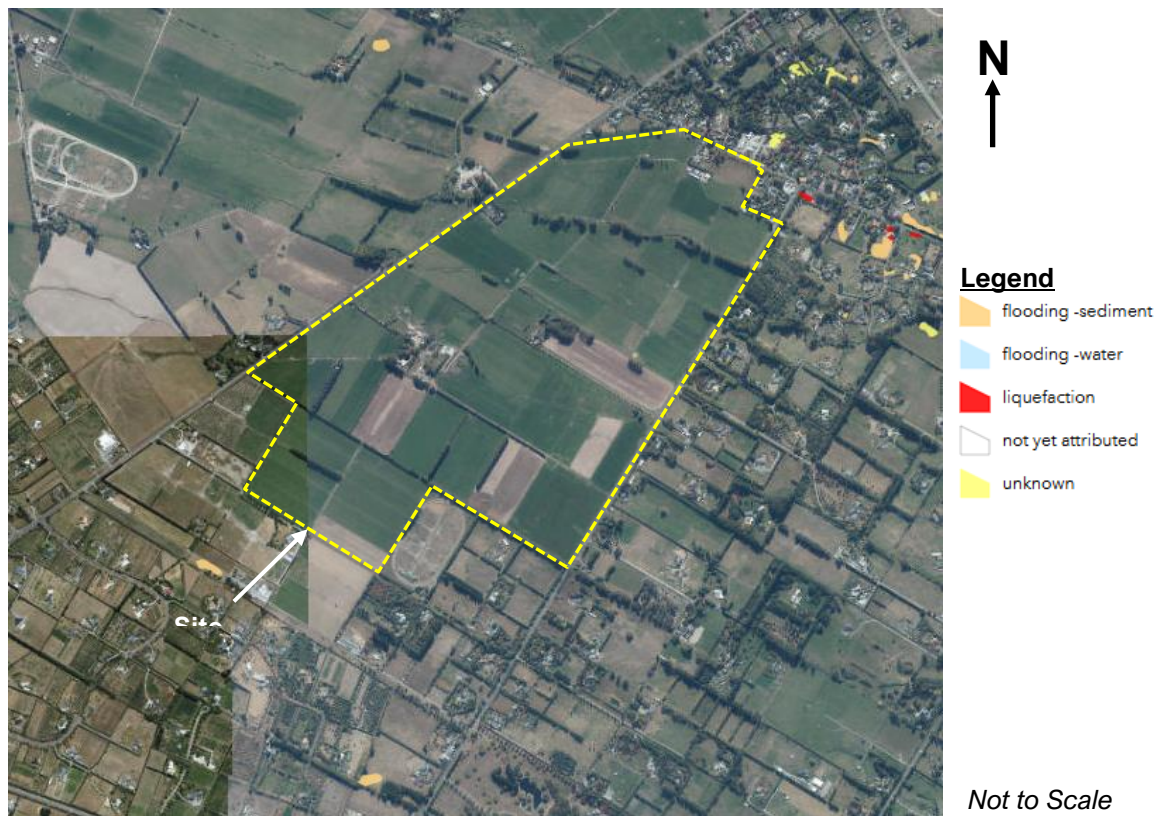


Figure 2: Environment Canterbury aerial and satellite imagery observation of inferred liquefaction following 22 February 2011 (data source Canterbury Maps)

11. The 4 September 2010 Mw 7.1 earthquake generated an estimated peak ground acceleration (PGA) of approximately 0.17g at the site (source Canterbury Geotechnical Database), resulting in two areas of observed liquefaction ejecta and surface flooding along the northern boundary of the site (refer Figure 1). The 22 February 2011 Mw 6.2 earthquake generated a peak ground acceleration (PGA) of approximately 0.13g at the site (source Canterbury Geotechnical Database). This resulted in minor localised flooding with sediment immediately south and northeast of the site, although no direct effects were recorded within the site itself (refer Figure 2). The occurrence of liquefaction and flooding, together with the relatively shallow groundwater levels and potential presence of liquefiable soil lenses, indicates that a residual liquefaction hazard exists at the site. I disagree with Tetra Tech Coffey's conclusion that liquefaction risk is "negligible". In my opinion, this conclusion is inconsistent with the observed site performance during the 2010-2011 Canterbury earthquakes and is not supported by a quantitative liquefaction assessment in the geotechnical report.
12. The MBIE Earthquake Geotechnical Engineering Practice Guideline: Module 1 (2021) identifies a serviceability limit state (SLS) design earthquake for geotechnical assessment with a PGA of 0.13g and magnitude Mw 7.5. For an importance level 2 structure with a 50-year design life, the ultimate limit state (ULS) design earthquake (1/500 annual probability of exceedance under NZS 1170.5) has a PGA of 0.35g and magnitude Mw 7.5. Based on a review of conditional PGA interpolation data for the Canterbury earthquakes (source Canterbury Geotechnical Database), ground shaking at the site during the 22 February 2011 earthquake was less severe than the SLS design event, while shaking during the 4 September 2010 earthquake was marginally greater than the SLS threshold. The recorded shaking was broadly equivalent to a probabilistic earthquake with an annual probability of exceedance between 1/25 and 1/50. Given that localised liquefaction and flooding were observed under these levels of ground motion, it is reasonable to conclude that the effects of a future ULS design earthquake would likely be more severe than those experienced during the 2010-2011 Canterbury Earthquake Sequence. This indicates that liquefaction hazard at the site warrants further quantitative assessment before it can be regarded as negligible.
13. While gravels generally exhibit elevated resistance to liquefaction triggering, liquefaction of gravelly soils can occur depending on relative density, particle size distribution, and the intensity of earthquake shaking. In addition, saturated lenses of gravelly sand, sand, or silt within the gravel deposits may be susceptible to liquefaction, depending on their insitu density and loading conditions. Site investigations typically encountered gravels at depths of approximately 0.4 m, with groundwater levels generally occurring at about 0.5 m below ground level. Given the shallow groundwater conditions, the potential presence of liquefiable soil lenses within the gravel unit, and observations of possible liquefaction-related effects during the Canterbury Earthquake Sequence, a quantitative liquefaction triggering assessment is recommended. Such an assessment should be undertaken using a representative selection of boreholes to characterise the liquefaction hazard at the site.
14. The Planning and Engineering Guidance for Potentially Liquefaction-Prone Land (MBIE, 2017) provides a framework for assessing low liquefaction vulnerability. Under this guidance, low liquefaction vulnerability is defined as a greater than 85% confidence that earthquake-induced ground damage will be "none to minor" during a design earthquake with an annual probability of exceedance of 1/500 (equivalent to a 500-year return period). Table 3.6 of the MBIE (2017) guidance indicates that a Level B liquefaction assessment is appropriate for the current consenting stage of the proposed Ōhoka subdivision. Table 3.3 recommends an average investigation density of 0.5 to 20 deep investigations per km², with a minimum of three deep investigations within each geological sub-unit. In my opinion, a Level B assessment should be completed to provide a quantitative assessment of liquefaction hazard and to demonstrate whether the site can be classified as having low liquefaction vulnerability in accordance with the MBIE guidance.
15. The Ōhoka development geotechnical assessment is based primarily on site-specific investigations comprising shallow subsurface explorations to depths of less than 3 m. Information on deeper subsurface conditions has been obtained from ten wells recorded in the Environment Canterbury (ECan) well log database and used by Tetra Tech Coffey in its assessment. These wells extend to depths ranging from approximately 9.8 m to 114 m and are generally located adjacent to the proposed subdivision. The well logs provide only simplified and generic descriptions of subsurface conditions, are inconsistent in their classification of materials, and contain no strength testing data. The logs indicate a range of materials, including gravels, sands, silts, and clays. While gravel appears to be the dominant material, the records also suggest the presence of interbedded sand and silt layers. The density and engineering properties of

these potentially liquefiable strata are unknown. In my opinion, the available data are insufficient to adequately characterise liquefaction hazard at the site. Additional subsurface investigations and testing would be required to define the distribution and density of potentially liquefiable soils and to support a robust quantitative liquefaction assessment.

- 16. I recommend that a minimum of three additional deep geotechnical investigations, extending to at least 15 m depth, be undertaken across the site. These investigations could comprise machine boreholes, Cone Penetration Tests (CPTs), and/or shear-wave velocity profiling. The investigation locations should be informed by observations of site and surrounding land performance during the Canterbury Earthquake Sequence to target areas most likely to contain potentially liquefiable soils and to support a quantitative liquefaction triggering assessment.

Information Gaps / Clarification Required

- 17. Undertake a minimum of three deep geotechnical investigations to depths of at least 15 m across the site. Investigations should comprise machine boreholes with Standard Penetration Testing (SPT) and engineering geological logging, Cone Penetration Testing (CPT), and/or shear-wave velocity profiling. The investigations should be designed to characterise subsurface conditions and provide the data necessary for a quantitative liquefaction assessment.
- 18. Undertake a quantitative liquefaction triggering assessment using the results of the site investigations to confirm the site's liquefaction vulnerability classification and assess the validity of adopting an equivalent TC1 foundation performance classification for future residential development.

Areas of Agreement

- 19. The proposed design does not include disposal of stormwater to ground. I agree that discharge to surface water bodies is preferable to infiltration, given the shallow groundwater conditions, upward groundwater gradients, and presence of springs in the area.
- 20. Batter slopes along waterways of 3H:1V are generally considered appropriate. However, the proposed 2m setback from the crest of the batter should be confirmed following assessment of the site's liquefaction vulnerability.
- 21. Earthworks will require stripping and separation of topsoil from the underlying soils. Reuse of site-derived gravelly materials as engineered fill is likely to be suitable, subject to appropriate excavation practices, material testing, and field quality assurance during placement.
- 22. I agree that subsoil drains installed adjacent to pavements would be beneficial to pavement performance. Subgrade performance and California Bearing Ratio (CBR) values are likely to be influenced by groundwater levels, local soil conditions, and earthworks practices.
- 23. The risk of static settlement is considered low due to the predominance of gravel soils across the site. Notwithstanding this, field verification testing is recommended to identify any localised soft zones that may affect building foundations or pavements.
- 24. 32. Based on the information reviewed, the site is appropriately classified as Subsoil Class D in accordance with NZS 1170.5.

Recommendations / Conditions

- 25. I recommend that consent conditions be imposed to address the following matters:
 - a. A liquefaction vulnerability assessment should be undertaken in accordance with Planning and Engineering Guidance for Potentially Liquefaction-Prone Land (MBIE, 2017).
 - b. Additional deep geotechnical investigations should be completed across the site to support a quantitative liquefaction assessment. These investigations should comprise a minimum of three investigations extending beyond 15 m depth and include in situ strength testing (e.g. SPT or CPT) and/or shear-wave velocity measurements.

- c. Based on the available information, the site appears suitable for development from a qualitative geotechnical perspective. However, a quantitative assessment of liquefaction hazard is required to adequately define the level of geotechnical risk and any associated mitigation measures.
- d. Further geotechnical investigation, assessment, and design should be undertaken to support the detailed design phase of the subdivision.
- e. Key geotechnical design assumptions for pavements, foundations, and earthworks should be verified during construction through appropriate inspection, testing, and quality assurance procedures.

Overall Position / Conclusions

26. In my opinion, geotechnical risks associated with the site can be appropriately managed through detailed design of foundations, earthworks, waterways, and associated infrastructure. However:
- a. Additional deep geotechnical investigations, including in situ strength testing and/or shear-wave velocity measurements, are required to support a quantitative assessment of liquefaction triggering and to assess liquefaction vulnerability in accordance with Planning and Engineering Guidance for Potentially Liquefaction-Prone Land (MBIE, 2017).
 - b. This assessment is particularly important given the shallow groundwater conditions, the potential for interbedded sand and silt lenses within the predominantly gravel soils underlying the site, and Environment Canterbury records indicating possible liquefaction-related effects observed in aerial and satellite imagery following the 2010-2011 Canterbury Earthquake Sequence.
 - c. The effectiveness of geotechnical risk management measures within the subdivision will depend on the detailed design, which has yet to be completed. Accordingly, the detailed geotechnical design should be subject to further review and approval as part of subsequent stages of the development process.

Yours sincerely



Marcus Gibson

Principal - Geotechnical Engineering

on behalf of

Beca Limited

Phone Number: +64-3-366-3521
Email: marcus.gibson@beca.com