IN THE MATTER Submission on SmartGrowth Strategy 2023-2073

STATEMENT OF EVIDENCE OF BENJAMIN O'LOUGHLIN ON BEHALF OF BELL ROAD LIMITED PARTNERSHIP

Dated 24 NOVEMBER 2023

---

1. QUALIFICATIONS AND EXPERIENCE

1.1 My full name is Benjamin O'Loughlin

1.2 I am an Engineering Geologist employed by ENGEO NZ Limited as an Associate Engineering Geologist and Tauranga Office Leader.

1.3 I am a Chartered Engineering Geologist and accredited TCC Category 1 Geo-Professional with 25 years professional practice experience as an Engineering Geologist.

1.4 I have been involved in the delivery of land development projects in the Bay of Plenty region for the past 15 years.

2. SCOPE OF MY EVIDENCE

2.1 My evidence addresses the following:

a) Background

b) Summary of ground conditions

c) Consolidation and creep settlement risk

d) Liquefaction risk

e) Embankment stability risk

f) Tsunami risk

g) Earthworks

h) Conclusion

3. BACKGROUND

3.1 The proposed 335 ha development, comprises a 129 Ha land parcel located between Tauranga Eastern Link and Bell Road (the site) and a potential additional 217 Ha being the Hurst Block, as set out in the Statement of Nathan York.

3.2 The proposal is to develop a mix of residential, and employment zones in what is referred to as the development zone. The area outside this will be used for wetlands.

3.3 As part of the stormwater approach, approximately 40% of the land area is to be constructed as a wetlands as set out in the Statement of Peter Moodie.

3.4 My work on the site to date has included review and preliminary geotechnical analysis of site investigation data (CPT, Testpits and geophysical survey) previously undertaken (by others) for the Bell Road site; installation of groundwater monitoring piezometers and ongoing monitoring (for a period of approximately 18 months); assessment of landform and geotechnical considerations for development provided in a preliminary geotechnical appraisal report.

3.5 ENGEO has provided a preliminary Geotechnical Assessment Report (PGAR) for the development zone covering the northern 129 Ha of the site. The report was prepared to guide future considerations for land development feasibility and planning.

3.6 Our assessment was also prepared adopting now superseded seismic values. Industry adopted seismic values have increased by approximately 30% since issue of the PGAR. Our analyses has been re-run to match updated guidance. The values presented within are reflective of current industry practice.

3.7 At the time of rezoning and development further geotechnical investigation and reporting will be carried out to inform master planning, consenting, detailed design and construction.

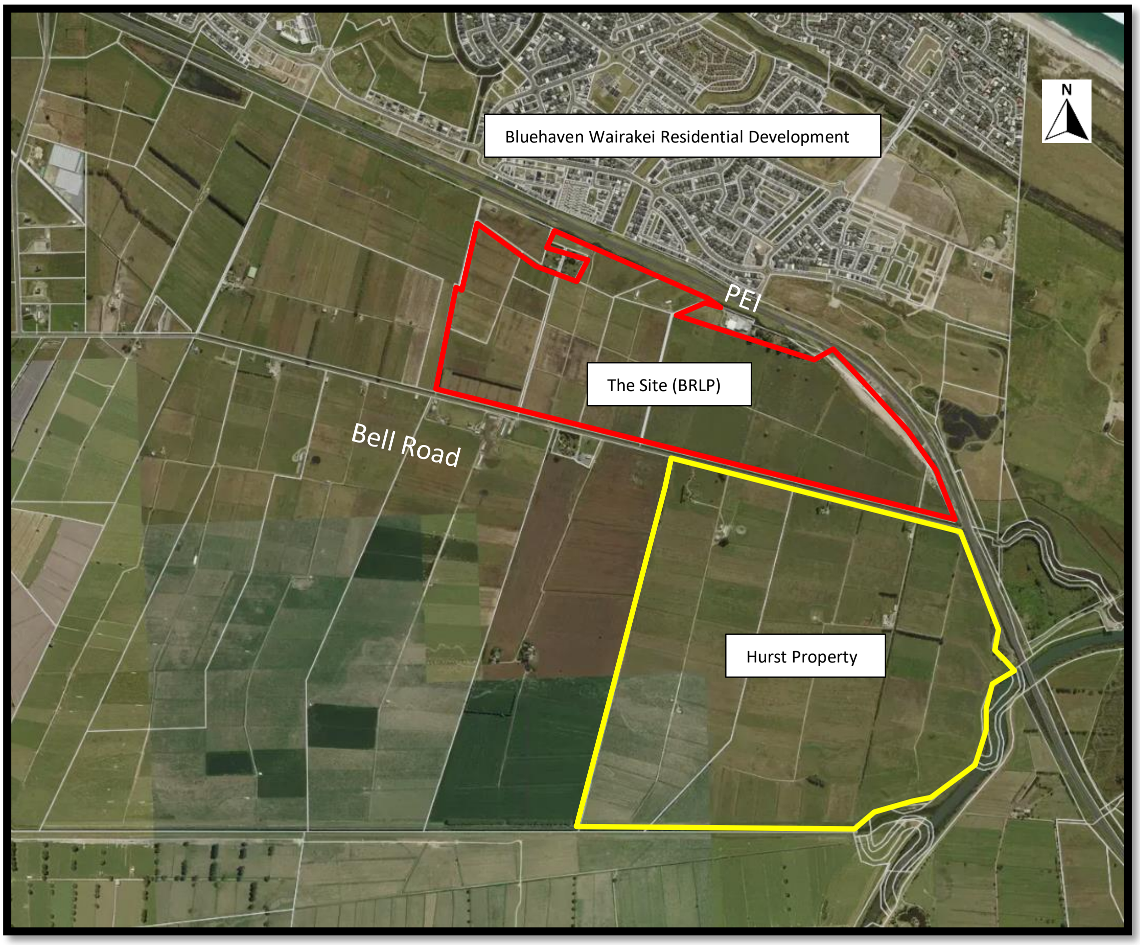

Figure 1: Site Locations, Sourced from GRIPS 15-11-2023

4. SUMMARY OF GROUND CONDITIONS

4.1 The site is generally underlain by a variable thickness of soft and compressible peat deposits (typically varying between 2-3 m depths), loose to medium dense liquefiable dune sands (5-8 m thick below the peat) and fluvially reworked deposits compromising laterally discontinuous stiff silts and fine to coarse loose sands at depth below the site.

4.2 Figure 2 shows the inferred peat deposit thickness across the site. The peat deposits show a general trend of increasing thickness towards the south (towards Bell Road) and east (towards the Kaituna River).

4.3 Along the northern boundary of the site, peat deposits are not present. These areas are underlain by surface sand dunes, typical of Papamoa area. The sand dunes rise in elevation from the flood plain (typ. 1 to 2 m RL) to approximately 5 to 6 m RL. The dunes sands dip to the south, beneath the surficial peat deposits. The lateral continuity of the sand dune deposits beneath the peat and south of Bell Rd is unknown at this stage.

4.4 Groundwater levels broadly dip with the landform to the southeast towards the Kaituna River and are typically within 0.5 m to 1.0 m of the ground surface across the flood plain.

4.5 We have been undertaking ongoing bi-monthly monitoring of groundwater levels across the site over the past 18 months. The groundwater monitoring to date shows the sensitivity of the groundwater table to rainfall, with an approximately 0.5 m increase in level following periods of heavy rainfall. Approximately 1.0 m of vertical fluctuation is observed across the 18 months of monitoring data.

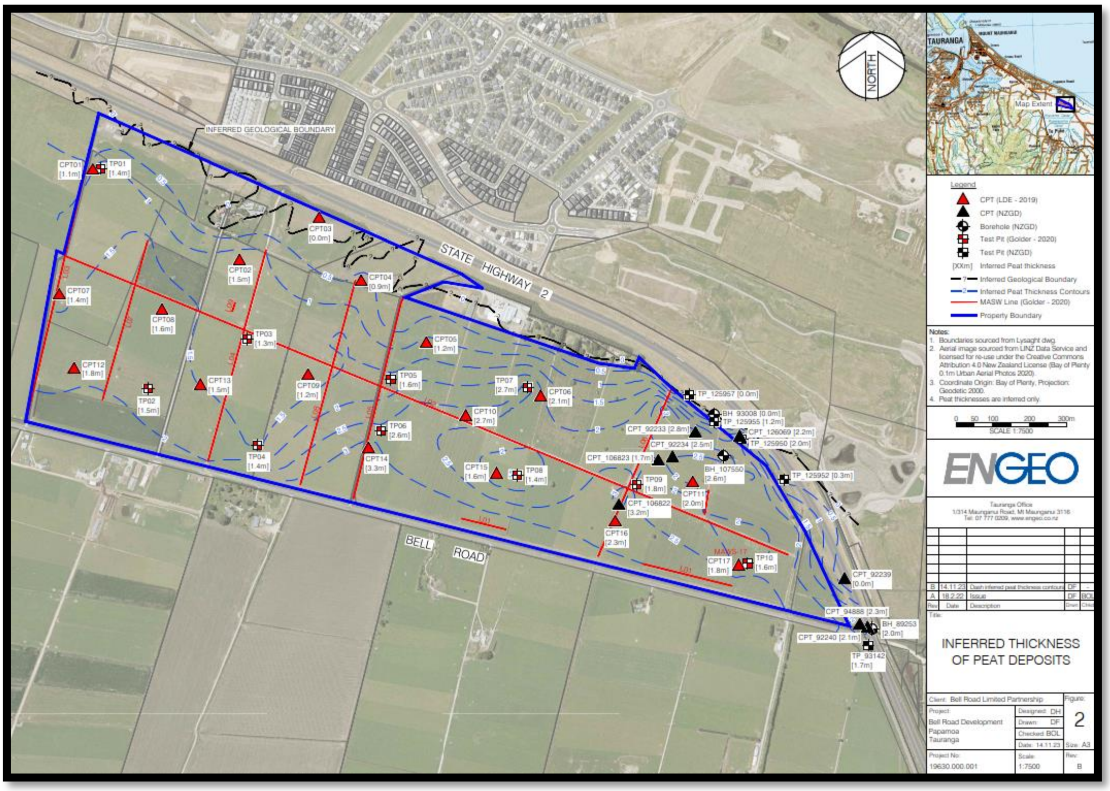

Figure 2: Peat isopach (thickness) map. Source from ENGEO 2022.

5. CONSOLIDATION AND CREEP SETTLEMENT RISK

5.1 Compressible peat deposits span the majority of the site (Figure 2).

5.2 When fill is placed to raise the site, these peat deposits will compress (consolidate) up to 70% of their thickness. The majority of this settlement will occur quickly (consolidation). Longer term (creep) settlement will also occur which will take many years.

5.3 To manage the consolidation and creep settlement risk, preloading with additional fill above the design level will be required over most of the development area.

5.4 Based on up to 4 m of peat thickness, and raising ground levels by 4 m, preliminary estimates of preload heights of between 2 - 4.5 m are anticipated to be required in order to reduce long term settlements to be less than 25 mm over 50 years. The placement of structural fill and preloading is expected to result in total settlements of up to 2 m, over an approximate duration of between 9 and 24 months.

5.5 Preloading to reduce long term settlement is used extensively across NZ for land and building projects. Locally this method has been adopted for commercial and industrial development at Tauriko Business Estate; and for residential developments around the Tauranga Harbour including at Judea, Bellevue and Bethlehem. Several lifeline roading networks have been constructed through preloading methodologies including the TEL, Route K and Takitimu Northern Link.

5.6 Where preloading to reduce settlements is not desired, alternative settlement mitigation options are available for the site, including the removal and replacement of peat deposits with engineering fill, and through deep ground improvement methods.

6. LIQUEFACTION RISK

6.1 Like the majority of Papamoa area the site is considered to be underlain by liquefiable soils.

6.2 Natural sands below the water table, underlying the peat (approximately 5 – 8 m thickness below the peat) and portions of the deeper fluvial deposits at depth (between approximately 12 and 16 m depth) are susceptible to the effects of liquefaction.

6.3 Our preliminary liquefaction analysis indicates that liquefaction is triggering between the 1 in 50 and 1 in 100 year earthquake events, with full liquefaction of the sands occurring between the 1 in 250 and 1 in 500 year earthquake events[1]. The ground response to large earthquake events (1 in 100 year, 1 in 2500 year and 1 in 3030 year) is similar to that predicted under the 1 in 500 year event (design ULS event).

6.4 Following earthworks to form the engineered landform, it is anticipated that the liquefiable deposits will be located at between 6 to 8 m below the finished surface. At this depth the effects of liquefaction are anticipated to be minor. As shown from the Canterbury earthquake sequence a non-liquefiable crust in the top 5 – 6 m of the ground surface

[1] Based on MBIE Module 1 Appendix A Peak Ground Acceleration for Te Puke. New Zealand Geotechnical Society (NZGS) and Ministry of Business, Innovation and Employment (MBIE) (2021). Earthquake geotechnical engineering practice. Module 1. Overview of the guidelines. Revision 1. November 2021.

suppressed the effects of liquefaction and its manifestation[2].

6.5 The 1 in 3030 year event[3] is required to be assessed as part of natural hazard assessment under the BOPRC RPS. Under this event, and considering the built landform, the expression of liquefaction is considered to be generally minor across the majority of the development. Moderate expression of liquefaction is predicted within the sand dune topography adjacent to the TEL.

6.6 Adoption of specific engineer design foundations to accommodate the magnitudes of predicted vertical deformation are expected to be required. This is now common practice for residential and industrial building development within the Papamoa area. Where sensitive infrastructure or buildings are required with a higher level of resilience (or lower levels of tolerance), deep ground improvement techniques can be implemented. This may be a requirement for lifeline infrastructure such as access routes, wastewater pump stations, water supply, and electrical sub-stations. Deep ground improvements may be designed to mitigate both liquefaction and settlement risks.

6.7 Based on the above, I am satisfied that the settlement and surface expression risks from liquefaction can be appropriately managed through standard and accepted engineering solutions for the site.

7. EMBANKMENT STABILITY RISK

7.1 A preliminary assessment of the stability of the proposed engineered fill embankments under seismic conditions has been completed. Two scenarios have been modelled, which include the stability of general embankments constructed over peat (typical scenario); and the stability of embankment in proximity to an excavated landform (in this preliminary case the Bell Road drain).

7.2 For the general embankment case, our analyses indicates that deep ground improvement is not required to maintain stability under seismic conditions however incorporation of a number of layers of geogrid along the edge of embankments within the structural fill and as basal reinforcement will be necessary. Preliminary displacements are predicted to be in the order of 100 mm within 50 – 100 m of the embankment edges during a ULS event.

7.3 For embankments in proximity to Bell Road (and other areas excavated below ground level) our analyses indicates that ground improvements are needed to prevent embankment failure under seismic conditions.

7.4 Preliminary ground improvement options include deep ground improvement such as deep soil mixing, placed in the ground along the margins of the excavations or beneath the toe of

[2] Cubrinovski M (2019). 'Some important considerations in the engineering assessment of soil liquefaction'. 2019 NZGS Geomechanics Lecture, Proceedings of 13th Australia New Zealand Conference on Geomechanics, Perth, Australia, April 1-3, 2019.

[3] 0.63g Peak Ground Acceleration adopting a $C_{0,1000}$ value of 0.41 and R_u value of 2.0.

the fill embankments, and designed to prevent failure and lateral movement of the embankment under the liquefied soil conditions.

7.5 Alternative methods may include shallow ground improvement options such as placement of shear keys or use of cohesive fills that will function to reduce but not eliminate, lateral movement. Preliminary analysis suggests lateral displacement may be in the order of 200 – 300 mm over 100 m. The magnitude of these displacements are expected to be addressed through specific foundation design.

7.6 Where ground improvement methods are not adopted, then building setbacks may be established to limit the risk of instability to future buildings and infrastructure.

7.7 Based on the above, I am satisfied that the seismic stability of the embankments can be appropriately managed through standard and accepted engineering solutions for the site.

8. TSUNAMI RISK

8.1 In 2011 GNS prepared a report modelling the Tsunami risk to Papamoa, Wairakei and Te Tumu and its implication for the SmarthGrowth Strategy[4]. It is noted that the site lies outside these specific areas, however the modelling considers the wider region. The greatest risk of tsunami is from rupture of the Kermadec Trench.

8.2 The majority of assessed scenarios show that the sand dunes along Papamoa and Te Tumu are high enough to prevent overtopping by the tsunami and therefore no inundation is modelled, this includes the most severe scenario modelled that assumes full rupture (1400 km length, Magnitude 9.4 earthquake, 540 year return period).

8.3 Inundation of the site is predicted for the worst case scenario (fault rupture from the Kermadec Trench extending to the south of the Hikurangi Margin) or variations of the Kermadec Trench rupture considering slip parameters from in the Tokohu Earthquake in Japan. These events overtop the Papamoa dunes and inundate the low lying area (including the site) behind the dunes flowing east towards the site and Kaituna River mouth (at Maketu). The Te Tumu sand dunes, which are 12 m high, are not overtopped during these modelled events.

8.4 Inundation depths for the site based on modelling of current ground levels are shown to vary between 2.0 m (4 m RL) at the western end of the site, to 1.5 m (2.5 m RL) at the east end of the site. Inundation on Hurst block is modelled to be generally 1 - 1.5 m (2.5 m RL). Parts of both the site and the Hurst block show areas of no inundation. Under this worst-case scenario, the Papamoa coastline is inundated by a 10 m high Tsunami.

8.5 We note that the areas of the site modelled to be inundated are proposed to be raised between 3.95 m RL to 6.95 m RL (average 4.1 m RL). The risk of Tsunami inundation from

[4] GNS Science Consultancy Report 2011/294, December 2011, Modelling of the tsunami risk to Papamoa, Wairakei and Te Tumu and implications for the SmarthGrowth Stategy

this modelled event is therefore anticipated to be greatly reduced by the changes to the elevation of the landform as part of the earthworks.

8.6 Modelling of the tsunami inundation impacts to the proposed landform may be necessary to demonstrate the impacts have been appropriately addressed as part of the natural hazards assessment required under the RPS at the time of a plan change to rezone the land.

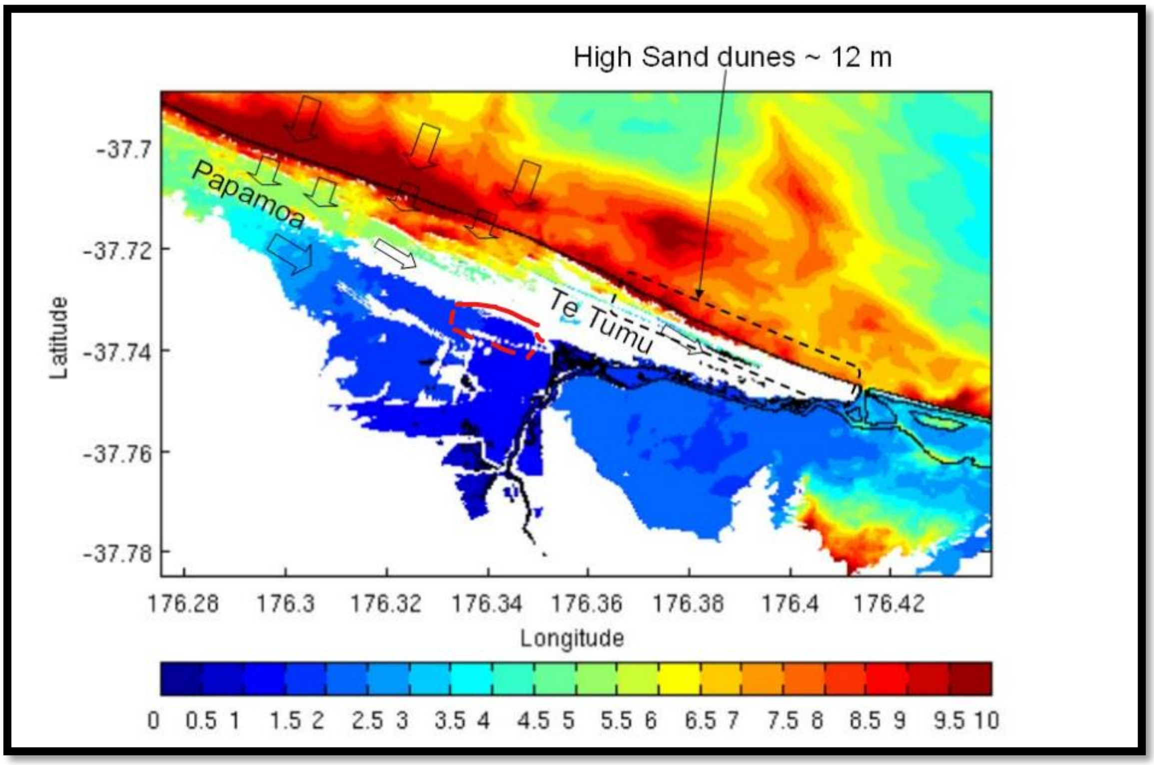

Figure 3: Tsunami Modelling Mw 9.4 earthquake (Full rupture of Kermadec) - Sourced from GNS Report 2011/294 December 2011. Approximate site shown in red dashed area

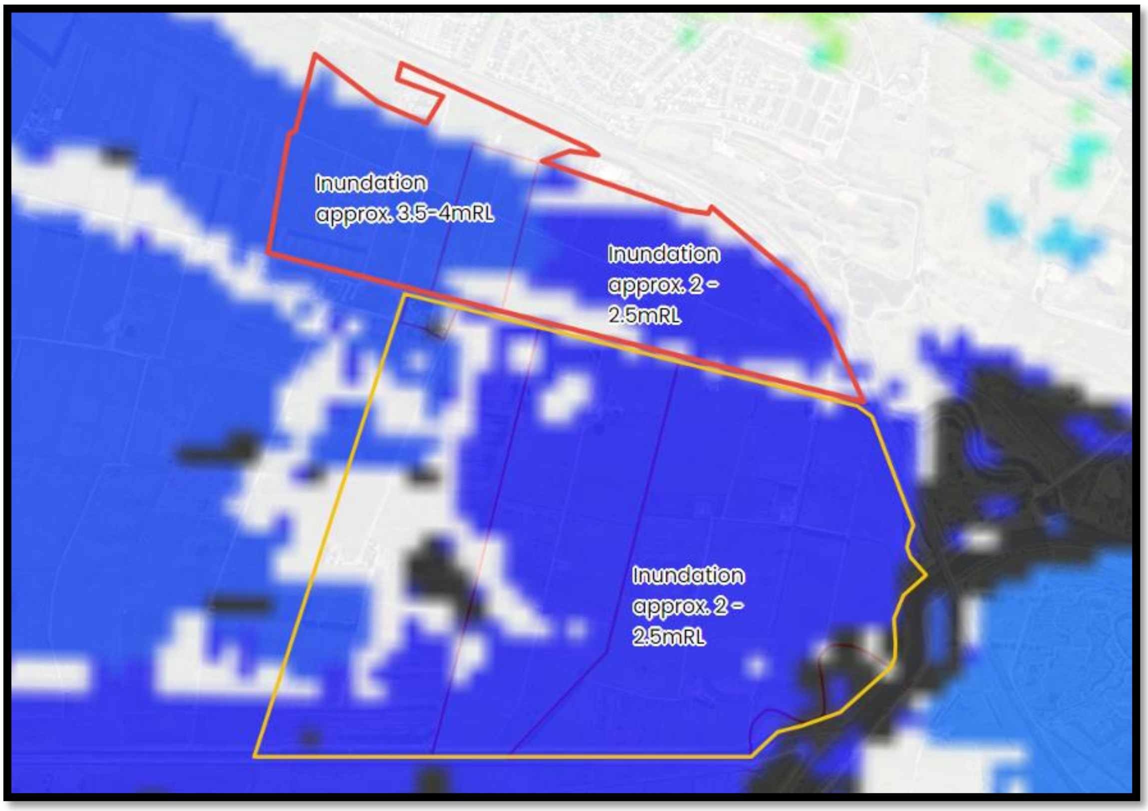

Figure 4: Tsunami Modelling Mw 9.4 EQ- Sourced from GNS Report 2011/294 December 2011. Site overlay with interpreted inundation levels (mRL).

9. EARTHWORKS

9.1 It is understood that the majority of the fill will be imported to site but there is also the opportunity to win fill from the site itself, (underlying sand) particularly during early stages of the development. General structural fills are anticipated to comprise imported pumice which is considered suitable for compaction. Its lower density will be favourable in reducing settlement magnitudes. Numerous pumice fill sources are available in close proximity to the site.

9.2 Cohesive soils are likely to be required to form wetland bunds and may be incorporated into shear keys or edge stabilisation measures. Suitable volcanic ash soils are available in the region.

9.3 Dune sands sourced from site are generally considered suitable for reuse as engineered fill as is my experience from a wide range of applications for land development within the Papamoa area. Given the elevated groundwater conditions, sand mining on the site will need to be through dredging type methods.

9.4 During construction, a staged fill approach will be required to maintain embankment stability during construction. The thickness of fill lifts and duration of hold time between subsequent lifts will be determined as part of detailed design and refined throughout construction with ongoing monitoring. This is considered standard practice for earthworks on soft ground to manage the risk of failure during construction.

9.5 Geogrid reinforcement will be required to be incorporated beneath the toe and edge of fill embankments to provide additional reinforcement during fill in order to maintain stability.

9.6 It is understood that development will incorporate creation of substantial wetlands to accommodate future stormwater requirements for the development. Geotechnical engineering will be necessary to inform stormwater earthworks design, particularly in respect of future bunds swales, and excavations, and associated impacts on the stability of the surrounding landform.

10. CONCLUSION

10.1 Geotechnical investigations completed to date confirm that the site is subject to a number of geotechnical constraints and geohazards including consolidation settlement, liquefaction, embankment stability, and tsunami. Based on the results of the preliminary assessment completed under my guidance, I consider that there are a number of appropriate, accepted and viable engineering solutions available to mitigate and / or manage the identified geotechnical constraints to accommodate the proposed future residential and employment zone land development.

10.2 This conclusion is subject to the completion of further geotechnical investigation and engineering involvement through all stages of design, construction observation and certification of landform, buildings and infrastructure. A Natural Hazards risk assessment will be required in accordance with the Bay of Plenty Regional Council Regional Policy Statement.