Before the Independent Hearing Panel
at Taupo

in the matter of:	Proposed Plan Change 36 to the Taupo District Plan – Request under Schedule 1 of the RMA to rezone Rural Land to Residential at Whareroa North by The Proprietors of Hauhungaroa No. 6
to:	**Taupo District Council**
Applicant:	**The Proprietors of Hauhungaroa No.6**

Statement of Evidence by Harshad Phadnis on behalf of The Proprietors of Hauhungaroa No.6

29 April 2020

1. INTRODUCTION

Name and qualifications

1.1. My full name is Harshad Sham Phadnis. I hold a Master of Science degree specialising in Geotechnical Engineering from the Georgia Institute of Technology in U.S.A. and a Bachelor of Technology degree in Civil Engineering from the University of Mumbai in India. I am a Chartered Professional Engineer in the practice field of geotechnical engineering and a Chartered Member registered with Engineering New Zealand (ID:1159638).

1.2. I have worked as a geotechnical engineer for Cheal Consultants Limited for the last ten months. Before this, I worked as a geotechnical engineer for CMW Geosciences in Auckland for one year and four months. Before moving to New Zealand, I have worked as a geotechnical engineer in the U.S.A. and Dubai (U.A.E). for a total of seven years and eleven months.

1.3. I have performed and managed numerous geotechnical investigations and construction inspections within the limits of Waikato Regional Council (since May 2019) and Auckland Council (since January 2018). I have also performed high level risk/hazard assessments and geotechnical analysis to support resource and subdivision consent applications and have performed detailed geotechnical analysis to support building consent applications. As a result, I am familiar with geotechnical issues commonly encountered in the subject site's geology e.g. issues related to pumiceous soils like difficulty in using conventional testing methods, change in fabric in case of over-compaction etc.

1.4. I confirm that I have read the "Code of Conduct for Expert Witnesses" contained in the Environment Court's Consolidated Practice Note 2014 and agree to comply with them in giving evidence in this proceeding. Except where I state that I am relying on evidence of another person, this written evidence is within my area of expertise. I have not omitted to consider material facts known to me that might alter or detract from the opinions expressed in this evidence.

1.5. I have been retained by the Proprietors of Hauhungaroa No.6 to provide geotechnical advice relating to the proposed plan change at Whareroa North.

1.6. As part of my engagement with the Proprietors of Hauhungaroa No.6, I have undertaken a visit to the site on 5 February 2020 and, in preparing this evidence, I have reviewed the following:

1.6.1 Edbrooke, S.W. (compiler) (2005). Geology of the Waikato Area. Institute of Geological & Nuclear Sciences.

1.6.2 Carryer, S.J. (22 April 1986). Report on the Geotechnical Aspects of the Whareroa Village Subdivision, Lake Taupo. Ref. 156/0764. Carryer & Associated Ltd.

1.6.3 Mitchell, M.T. (19 October 2006). Site Assessment and Supplementary Geotechnical Engineering Appraisal – Proposed Whareroa North Residential Subdivision – Hauhungaroa No. 6, Whareroa Road North, West Lake Taupo. Ref. T – 9036/1. Mark T Mitchell Limited.

1.6.4 Martinez, A. (18 October 2018). Verification of Geotechnical Constraints for Residential Development. Ref. IBA 1070L150. Cheal Consultants Limited.

1.6.5 Kelly, T. (26 September 2019). Preliminary Stormwater Assessment. Ref. IBA 1070 Rev. 4. Cheal Consultants Limited.

1.6.6 Phillips, M. and Gray, I. (31 March 2020). Proposed Plan Change 36 – Whareroa North – Initial Geotechnical Review. Ref. 2-37780.00. WSP Opus.

2. EXECUTIVE SUMMARY

2.1. My evidence is specific to the matters of geotechnical engineering and is based on my experience in the field of geotechnical engineering, review of reports identified in Section 1.6 and the site visit conducted on 05 February 2020.

2.2. I have performed a desktop assessment to identify the geo-hazards that can potentially affect the site. A summary of this has been presented in the table in Section 9.2. I consider that instability, liquefaction susceptibility, lateral spreading, flow liquefaction, compressible soils, settlement/ subsidence including differential settlements, piping/underground erosion, effects and/ or appropriateness of onsite soakage effects on the "bowl" and the scar, are all potential geo-hazards that can affect the development site and the access corridor.

2.3. Comprehensive geotechnical investigations are required before applying for subdivision consent. A draft outlining the geotechnical investigations that are proposed as part of the Preliminary Stage is presented in Appendix 12. These investigations will enable us to determine whether any of the geo-hazards listed in the above table do pose a risk to the development and to provide solutions if they do.

2.4. The vegetation removal for earthworks needed for the comprehensive geotechnical investigations will need resource consent because some of the work is in a Significant Natural Area. In my view there are practical reasons related to project management why the further geotechnical investigations should happen after the plan change is confirmed rather than before. I am confident that there is sufficient geotechnical information presently available to support the change for residential development.

2.5. All likely geo-hazards have been encountered previously to varying degrees in and around Taupo and engineering solutions exist to mitigate the effects of these. The table presented in Section 12.1 summarises typical and worst-case scenario and related mitigation solutions which are used routinely by professional engineers in the area and have a proven success record. Hence, even if the worst-case scenario is encountered, I consider, that the site and access corridor is or can be made to be suitable for residential development from a geotechnical perspective.

2.6. In summary, based on my assessment and in the words of the Waikato Regional Policy Statement (WRPS) policies on natural hazards, it is my opinion that the land affected by the Whareroa North proposal is not a *"primary hazard zone"* (being *'an area in which the risk to life, property or the environment from natural hazards is intolerable')* and the proposal will not create an "*intolerable risk*" (being a *'risk which cannot be justified and risk reduction is essential e.g. residential housing being developed in a primary hazard zone').*

3. PLAN CHANGE 36: WHAREROA

3.1. I have been involved with this project since 5 February 2020 when I was briefed about the development of Whareroa over the decades and this project, in particular, by David Forsyth, who is Cheal Consultants Limited's Managing Director and a registered professional surveyor, and Michael Keys, the Managing Director of KeySolutions who is a consultant engineer. I then visited the site on the same day. These activities were performed so that I can provide evidence for the proposed plan change.

3.2. I understand that Cheal Consultants Limited has provided engineering support related to civil and geotechnical disciplines for this project since 2009. None of the geotechnical engineers and engineering geologists who have worked on this project over the past years work for Cheal Consultants Limited any longer and hence, I will provide evidence for the hearing on the proposed plan change.

3.3. Andres Martinez, a geotechnical engineer who worked for Cheal Consultants Limited, visited the site on 25 September 2018 and verified the geotechnical constraints for residential development at the site. His findings have been summarised in a letter report which was issued on 18 October 2018 which is discussed in Section 7. I attach a copy of that report for ease of reference as Appendix 1.

4. GEOLOGICAL BACKGROUND OF THE AREA

4.1. The geological setting of the area based on GNS Science's New Zealand 1:250,000 GIS map has been presented in Appendix 2. Low-lying areas of the proposed access road are anticipated to be underlain by Holocene river deposits comprising of pumice sand, silt and gravel alluvium with charcoal fragments derived from Taupo Pumice Formation. The elevated sections of the proposed access road where cuts and batters are anticipated are likely to be underlain by Taupo Pumice Formation ignimbrite and by Oruanui Formation ignimbrite. The proposed lots are anticipated to be underlain by Oruanui Formation ignimbrite which are anticipated to be covered by aeolian tephra from the Taupo eruption that occurred in 181 A.D.

4.2. As per Ref. 1.6.1, both Taupo Pumice Formation ignimbrite and Oruanui Formation ignimbrite are the youngest ignimbrite deposits in the Waikato region and are related to the Oruanui eruption that occurred approximately 26,500 years ago and the Taupo eruption that occurred in 181 A.D. Both Taupo Pumice Formation ignimbrite and Oruanui Formation ignimbrite comprise of non-welded fine-grained ignimbrite and some reworked deposits.

4.3. The New Zealand Geotechnical Database (NZGD), which is a free-to-use online database of geotechnical information funded by the Ministry of Business, Innovation & Employment, was checked for geotechnical information in the vicinity of the site. As seen in Appendix 3, the nearest geotechnical information on NZGD is approximately 30 kilometres away and due to the distance, this information was not used in this statement of evidence.

4.4. An internet search was also performed to see if any publicly available geological and/or geotechnical information is available. None was available.

5. CARRYER & ASSOCIATES' REPORT (22 April 1986)

5.1. Carryer & Associates Ltd. visited the site on 21 April 1986, carried out a walk over survey, and inspected the topography and surface material. A report, Ref. 1.6.2, which discusses observations made during the walk over survey, engineering geology and implications on the then proposed subdivision has been presented as Appendix 4.

5.2. As per Ref. 1.6.2, the materials at the site are stable and no reason was observed to suggest that the proposed development would affect the stability in any way.

5.3. The only potential problem that was identified was erosion resulting from the concentration of stormwater, particularly on the steeper slopes, however, the author also mentions that disposal of stormwater in to the ground would be effective over most of the area except for areas close to the crest of steep slopes i.e. within 20m from the crest where detailed consideration of stormwater and geotechnical details will be required.

5.4. Detailed engineering design will be required for steep cuts with slopes in excess of 15 degrees to ensure their stability.

5.5. Ref. 1.6.2 concludes that the proposed development conforms with challenges posed by the geology of the area and that the proposed development will enhance the stability of the area except for a small area within 20m from the crest of steep slopes. Specific control of stormwater disposal and geotechnical design will be required in this area to avoid any detrimental effects.

6. MARK T. MITCHELL'S REPORT (19 October 2006)

6.1. Mark T Mitchell Limited performed investigations to determine the stratigraphy at the low-lying area which has been referred as the "bowl" (as shown in Appendix 8) and determine special conditions required, if any, if the "bowl" is to be filled. The results including geotechnical logs from seven test pits performed up to 4.8 metres below the ground level when the test was conducted (mbgl) are summarised in Ref. 1.6.3 which has been presented as Appendix 5. Filling the "bowl" is no longer a part of the proposal but this change was not driven by any geotechnical issue(s).

6.2. Ref. 1.6.3 is a supplementary report (dated 19th October 2006) to a report issued on 4 August 2006 that included site testing data and recommendations for on-site stormwater disposal. As the August report was not available, it does not form part of this review.

6.3. Based on photos presented in Ref. 1.6.3 and terrain observed when I visited the site in February 2020, I consider that no manmade changes have been carried out on the "bowl" and all long-term geological changes are anticipated to be slow occurring.

6.4. Ref. 1.6.3 concludes that the "bowl" is a natural bench feature created by welded ignimbrite bedrock being mantled by younger aeolian tephra which has undergone some down-slope movement which is likely to have occurred after the tephra were deposited i.e. approximately around 181 A.D. The aeolian tephra is likely to be compressible, and this is discussed further in Section 9.8 - 9.13 of my evidence. The October 2006 Mitchell Report also concludes that the "bowl" area is stable.

7. CHEAL GEOTECHNICAL REPORT (18 October 2018)

7.1. Cheal Consultants Limited issued Ref. 1.6.4 after receiving a Request for Further Information (**RFI**) from the Taupo District Council. Ref. 1.6.4 summarises the assessment of geotechnical constraints potentially affecting the land for the proposed development and addresses the concern related to stormwater mitigation as raised in the RFI.

7.2. The scar was tracked back to 1969 A.D where it is first seen in photographs. The scar has potentially been formed by an ephemeral drainage path which saturated the loosely packed material at the base which were then eroded by the former course of the Whareroa Stream over time. This then undermined the upper soils i.e. soils near the crown of the scar, and this process continued to form the scar. This is anticipated to extend back (i.e. to the north) by at least 15m before reaching its equilibrium position and hence, any stormwater pond(s) (including the maximum extent of batters) should be beyond this distance. The process is anticipated to be longitudinal i.e. from south to north and not lateral.

7.3. Ref. 1.6.4 concludes that surface runoff from the “bowl” has a low potential to incise the scar, however, infiltrated runoff creating underground flow paths is likely to (if allowed to continue) incise the scar and affect any proposed development located within the “bowl”. I believe that careful design of the stormwater management system for the development area (once the geotechnical investigations detailed later in my evidence are complete) can prevent this from occurring.

8. PRELIMINARY STORMWATER ASSESSMENT (26 September 2019)

8.1. Cheal Consultants Limited issued Ref. 1.6.5, which has been presented as Appendix 6, after receiving the RFI from the Taupo District Council. Ref. 1.6.5 broadly outlines the proposed approach for dealing with stormwater flows and quality at the site. The implications of this stormwater methodology as it relates to geotechnical engineering are covered in this section.

8.2. The general methodology for stormwater mitigation is that drilled soak holes will be provided for the roads to dispose of stormwater from a 10% annual exceedance probability (AEP) event and stormwater from a 1% AEP event will be conveyed via the road reserve to an attenuation pond. The primary purpose of the attenuation pond will be to minimise peak flow rates, to reduce concentration, and to divert flow away from the scar. Discharge from the pond will be via sheet flow which will be restricted to the pre-development 10% AEP flow rate. At this stage, it is envisaged that pond 1 will be lined and ponds 2 and 3 will not be lined but a final decision regarding this aspect of the work will be made once further geotechnical investigations and design activities are completed.

8.3. Lots will discharge stormwater to ground via soak pits or infiltration trenches. Lots within post-development Soakage Area 1 will have above-ground stormwater tanks installed at the time of building to attenuate the 10% AEP storm and restrict soakage outflows to pre-development flow rates.

8.4. Bioretention swales have been proposed in Ref. 1.6.5 and I do not anticipate any geotechnical problems with having vegetated bioretention swales treating road runoff and having sheet flow discharge remote from the scar.

9. TECHNICAL DISCUSSION

9.1. Firstly, in addition to the investigation work described in Sections 5, 6 and 7 of my evidence, some cone penetration tests (CPT) were performed in 2010 by Geotech Drilling, a drilling company which specialises in performing geotechnical and environmental investigations. The test locations were plotted using multiple coordinate systems. Interpreted coordinates and associated test locations presented in Appendix 7 are deemed to be the best interpretations of CPT locations. The CPT test records are also presented in Appendix 7. As the CPTs were performed approximately ten years ago, there is a degree of uncertainty regarding the test locations, and because interpreting soil types using CPT data is not reliable in pumiceous soils, the CPT records have not been relied upon in preparing this evidence.

<u>Hazards Assessment</u>

9.2. I have performed a desktop assessment to identify the geo-hazards that can potentially affect the site. A summary of this has been presented in the table below. I consider that instability, liquefaction susceptibility, lateral spreading, flow liquefaction, compressible soils, settlement/ subsidence including differential settlements, piping/ underground erosion, effects and/ or appropriateness of onsite soakage, effects on the "bowl" and the scar are all potential geo-hazards that can affect the development site and the access corridor. However, I emphasise that all of these geo-hazards have been encountered previously to varying degrees in and around Taupo and engineering solutions exist to mitigate the effects of these. These solutions are used routinely by professional engineers in the area and have a proven success record.

<table>
<tr><th>Hazard</th><th>Assessment</th></tr>
<tr><td>Inundation/ Groundwater Flooding</td><td>The Whareroa Stream and Lake Taupo levels are approximately RL 360m and RL 358m respectively. These are the only water bodies that can affect the proposed development.

The proposed bridge will be designed to have an appropriate waterway cross-section to avoid inundation/ flooding from at least the 1% AEP stream flow.</td></tr>
<tr><td>Expansive Soils</td><td>As per Section 4, the proposed lots are anticipated to be underlain by pumiceous soils and further by ignimbrite of Oruanui Formation. Some clayey silts were identified on site (ref 1.6.3) but this report does not identify expansive behaviour to be a risk.

Additionally, expansive behaviour of fine-grained soils will be assessed by performing laboratory testing which will be undertaken prior to the resource/subdivision consent stage. If any expansive soils are encountered, appropriate recommendations will be provided e.g. deeply embedded foundations, structures that can tolerate seasonal movement of soil etc.</td></tr>
<tr><td>Subsidence including differential settlement</td><td rowspan="2">Low-lying areas of the proposed access road are anticipated to be underlain by Holocene river deposits comprising of pumice sand, silt and gravel alluvium with charcoal fragments derived from Taupo Pumice Formation. The elevated sections of the proposed access road where cuts and fill batters are anticipated are likely to be underlain by Taupo Pumice Formation ignimbrite and by Oruanui Formation ignimbrite. The area where the lots are proposed is anticipated to be underlain by Oruanui Formation ignimbrite which is anticipated to be</td></tr>
<tr><td>Ground settlement due to compressible soils including differential settlement</td></tr>
</table>

Hazard	Assessment
	covered by aeolian tephra from the Taupo eruption. As mentioned above, some silts were encountered during investigations performed by Mark T Mitchell Limited in 2006. Some stream or lake related deposits can also be expected near the bridge abutments. All of these will be identified during further comprehensive geotechnical investigations (detailed later in my evidence). As mentioned in 9.2 above, if subsidence or settlement (including differential settlement) due to compressible soils are anticipated, appropriate solutions are available. These include stripping of compressible soils, deeply embedded foundations, pre-loading of soils, structural design to tolerate settlements etc.
Corrosive Soils	Based on the geology at the site which has been presented in Section 4, we do not anticipate corrosive soils at the site.
Slope Stability	The proposed lots will be on flat to gently sloping ground except around the edges of the "bowl" which is moderately steep. Slope stability is not expected to be a problem. The design of cuts (including benching) and fills to form the access road, bridge approaches and the slope along the southern edge of the proposed subdivision will take into account the findings of the further geotechnical investigations.
Erosion – river, lakeshore, wind, etc	Lake Taupo is not anticipated to cause erosion related issues for the proposed bridge or bridge approaches as it/they are sufficiently upstream from the lake itself. The bridge abutments will be designed to sustain stream-induced erosion. Overland flow at the proposed lots will be appropriately engineered as a part of the stormwater detailed design. Hence, I do not consider erosion to pose a risk to the proposed development.
Internal/ underground erosion (including tomo formation)	Topographical information and the site visit undertaken on 5 February 2020 did not show concentrated watercourses or blind gullies onsite. As per Ref. 1.6.4, underground flow paths are likely to affect the proposed development. Underground flow paths will be located and assessed by performing deep investigations including machine-drilled boreholes as detailed later in my evidence. If underground flow paths are identified, the long-term stability will be mitigated, and the stormwater management system will be designed to address this potential hazard.
Geothermal eruptions Geothermal gas Geothermal subsidence	The site is not anticipated to be in a geothermal area. Geothermal eruptions, geothermal gas and geothermal subsidence are not anticipated to be markedly different from the wider Taupo region and hence are not considered to be a risk.
Volcanic eruptions	The volcanic activity is not anticipated to be markedly different from the wider Taupo region and hence is not considered to be a risk.
Soil Contamination	This will be assessed at the further preliminary investigation stage.
Liquefaction, lateral spreading and flow liquefaction	The proposed lots are anticipated to be underlain by pumiceous soils and further by ignimbrite of Oruanui Formation. Some loose sands may exist, but the proposed lots are on an elevated terrace ranging approximately from RL 395m to RL 420m. I expect the groundwater level to be controlled by the Whareroa Stream at approximately RL 360m. The groundwater level under the proposed lots will be approximately 35m deep. This will be confirmed in the further geotechnical investigations. Therefore, saturated conditions (which are

Hazard	Assessment
	necessary to cause liquefaction) are not expected in the top 35m of the soil profile so liquefaction is not anticipated to occur in this area. Effects of any liquefaction below this depth will be negligible. The area of the proposed bridge abutment is considered more likely to be susceptible to liquefaction, lateral spreading and flow liquefaction effects based on the shallow groundwater table controlled by the nearby Whareroa Stream. Machine-drilled borehole and cone penetration testing as well as laboratory testing will be performed near the bridge abutment to assess liquefaction, lateral spreading and flow liquefaction potential. If liquefaction, lateral spreading and flow liquefaction effects are assessed to pose a risk, appropriate solutions (e.g. founding on a non-liquefiable layer, using drainage to control the groundwater level, performing ground improvement to avoid liquefaction etc) will be incorporated into the detailed design.
Fill Material	No fill material was observed during the site visit undertaken on 5 February 2020. No fill material is anticipated at the site based on the site's historical use as a farm.
Historic lake or stream beds and lake terraces	Lake/ stream deposits at the proposed subdivision, along the proposed approach road and at the proposed bridge and its abutment will be assessed during the further geotechnical investigations. If such deposits are revealed, appropriate solutions (e.g. stripping of these deposits, deeply embedded foundations, appropriate structural design etc) will be incorporated into the final design.

9.3. In summary, based on my assessment and in the words of the WRPS policies on natural hazards, it is my opinion that the land affected by the Whareroa North proposal is not a *"primary hazard zone"* (being *'an area in which the risk to life, property or the environment from natural hazards is intolerable')* and the proposal will not create an "*intolerable risk*" (being a *'risk which cannot be justified and risk reduction is essential e.g. residential housing being developed in a primary hazard zone')*.

<u>Geotechnical Investigations</u>

9.4. As already mentioned, further comprehensive geotechnical investigations are required before applying for subdivision consent. A draft outlining the geotechnical investigations that are proposed is presented in Appendix 12. These investigations will enable us to determine whether any of the geo-hazards listed in the above table do pose a risk to the development and to provide solutions if they do.

9.5. In an ideal scenario, geotechnical investigations would have been performed at the plan change stage as per Table 2.1 of Module 2 of the Earthquake Geotechnical Engineering Practice which was published to explain current practice in earthquake geotechnical engineering. This document is a guideline and is not a national code. It should be noted that performing comprehensive geotechnical investigations will require some vegetation to be cleared and tracks being established using diggers and other construction equipment. Some areas, particularly along the proposed access road and near the bridge, are classified as Significant Natural Areas (SNA) and hence will require a resource consent to be granted before any activity is undertaken in those areas because vegetation will be disturbed. The site is located away from major centres and the investigation campaign is anticipated to last twenty to thirty working days. Hence, we consider it most reasonable to perform all investigations in one campaign instead of performing investigations over multiple campaigns to minimise the disturbance, cost and effort to establish cleared areas, create tracks and mobilise construction equipment, geotechnical rigs and personnel. The option of performing one machine-drilled borehole and four cone penetration tests near the “bowl” as suggested by Taupo District Council’s geotechnical expert was considered. Performing only one machine-drilled borehole and four cone penetration tests will not provide information across the entire site as the soil strata is anticipated to vary to a certain degree across the entire site. Thus, we cannot discuss the effects of geo-hazards across the entire site based on one machine-drilled borehole and four cone penetration tests. Hence, I recommend that all geotechnical investigations be performed as one campaign during the Preliminary Stage after the plan change request is approved.

9.6. As discussed in 9.2 above, I believe that any geo-hazards that become apparent during the further geotechnical investigations are likely to have been encountered in and around Taupo in the past and engineering solutions exist to mitigate the effects of these. I therefore don’t believe that it is necessary to undertake the investigation work outlined in Ms Phillips evidence prior to the Plan Change process.

9.7. We propose performing deep machine-drilled boreholes as well as CPTs to provide information about deep stratigraphy at the proposed dwelling areas, along the access road and near the bridge. Dynamic cone penetrometer (Scala) tests are proposed to provide an indication of the in-situ California Bearing Ratio (CBR). Soakage tests and in-situ permeability tests are proposed to check the available soakage rates. Groundwater levels will be monitored in piezometers installed in some of the machine-drilled boreholes for a period of at least three months to get a better understanding of the groundwater table.

<u>Scar Feature and Stormwater Design</u>

9.8. With respect to the scar at the lower end of the “bowl”, this could have been caused either by surface runoff (as per Ref. 1.6.4), by underground flow paths as explained in Section 6.3, or by a combination of these. I believe the better stormwater management that will be incorporated into the development (compared with the status quo) will arrest this situation.

9.9. Based on a review of Ref. 1.6.5, I consider that there won’t be any surface runoff through the scar. Stormwater from a 1% AEP event will be conveyed via the road reserve to an attenuation pond which will discharge from the pond via sheet flow that will be restricted to the pre-development 10% AEP flow rate well away from the scar as mentioned in Ref. 1.6.5. Additionally, surface runoff has a low potential to incise the scar as per Ref. 1.6.4. Based on high friction angles of pumiceous material and no major instabilities being observed away from the scar, it is considered that the ground between the pond and the bank will be stable and be able to accommodate periodic sheet flows from the pond. Hence, I consider that worsening of the scar due to surface runoff and instability of the ground between the pond and the bank will not occur and this scenario is referred as the typical scenario. These details will be assessed as a part of the proposed further geotechnical investigations and analysed before the subdivision consent application is submitted. Should ground conditions dictate, the outflow swale/sill lengths can be increased so that sheet flow occurs over a larger ground area thus reducing erosion risk. Alternatively, the ponds can be relocated and sheet flows can be diverted over areas which are stable.

9.10. As per Ref. 1.6.5, stormwater from a 10% AEP event will be through drilled soak holes as shown on drawing number IBA1070-656 Rev. B dated 11 September 2019 included in Ref. 1.6.5 and the nearest soakage hole is located at a minimum distance of 75m from the scar. Based on my knowledge of the geology at the site, the proposed design, I consider that the development of the land will be beneficial rather than a threat to the scar as there will not be any surface runoff through the scar and proposed soakage is remote from this feature. The scenario I have described is the "expected" situation. Should the further investigations suggest that there is a possibility of increasing underground flows, the design will be modified accordingly.

9.11. If underground flow paths are encountered during the further investigations, there should be no on-site soakage in the pre-developed Catchment B as this infiltration could aggravate the situation at the scar as identified in Ref. 1.6.4. This scenario is considered as the worst-case scenario.

9.12. I infer that observations made at and near the "bowl" and the scar are primarily because there are no mitigation measures in place at present. I believe that machine-drilled borehole information will assist us to assess whether underground flow paths are possible and a decision can be made to either use or avoid soakage holes in that area. Both of these scenarios will provide an engineered solution to reduce the erosion at the scar. If the site is left in its present state, the erosion will continue. The development of the land is hence considered to be beneficial with respect to arresting the erosion at the scar.

9.13. The option of piping stormwater to the Whareroa stream to provide outlets for the ponds was discussed by Cheal's civil engineer (Tony Kelly) and geotechnical engineer (Andres Martinez) but not pursued due to the potential impacts on the stream including water quality degradation and erosion as well as the possibility of water flowing along the outside of the pipe and causing subsurface erosion. This approach is also contrary to current best industry practice and cultural expectations.

Liquefaction

9.14. With respect to potential liquefaction, the proposed lots are anticipated to be underlain by pumiceous soils and further by ignimbrite of Oruanui Formation. Some of the materials might be loose sands which are considered to be susceptible to liquefaction. As discussed in the table in 9.2 of my evidence, the proposed lots will be located on an elevated terrace ranging approximately from RL 395m to RL 420m. I expect the groundwater level to be controlled by the Whareroa Stream at approximately RL 360m. The groundwater level under the proposed lots will likely be approximately 35m deep. This will be monitored by measuring groundwater levels as a part of the further investigations before the resource/subdivision consent application is submitted. Hence, saturated conditions which are necessary to cause liquefaction are not anticipated up to 35m below the proposed lots i.e. liquefaction is not anticipated to occur up to a depth of 35m below the proposed lots. Effects of any liquefaction below this depth will be negligible. Only the general area of the proposed bridge is considered to be potentially susceptible to liquefaction, lateral spreading and flow liquefaction effects due to the shallow groundwater table controlled by the nearby Whareroa Stream. Machine-drilled borehole and cone penetration testing as well as laboratory testing will be performed near the bridge abutment sites to assess liquefaction, lateral spreading and flow liquefaction potential. If these effects are assessed to pose a risk, appropriate design mitigation will be applied.

<u>Stability</u>

9.15. Dr. Ruth & Simon Ewen, Ian Sutcliffe, Robert & Jo Colman and Michael Townson Miller have submitted their concern about the proposed access road passing through an "unstable" area with evidence of slips. I consider that the general stratigraphy through the area where the access road is proposed will be colluvium underlain by silt/sand. In my view, only the upper layers are susceptible to shallow slope failures (as seen in Appendix 9) when they are not held together by grass or tree roots and/or during rainfall events. I consider that "unstable ground" and/or "land slip" identified by submitters is likely to be the result of these shallow slope failures. These will be underlain either by Taupo Pumice Formation ignimbrite and Oruanui Formation ignimbrite. Based on my experience of pumiceous soils in and around Taupo, I consider that pumiceous soils/ignimbrite can stand up very well at slopes (batters) of 1:4 (1 horizontal to 4 vertical) (refer Appendix 10). If the cut heights are greater than 5m, then benching (terraces), with a minimum width of 3m, should be utilised. This is considered as the typical scenario. A three-dimensional representation of cuts and fills involved for the proposed access road is presented in Appendix 11. Maximum cut height of 5m is shown with a bench width of 3m. The fill batters in the vicinity of the bridge are shown at slopes (batters) of 1.5:1 (1.5 horizontal to 1 vertical) which I consider to be appropriate for the expected (typical) scenario.

9.16. If the machine drilled boreholes along the access corridor (discussed in Section 9.7) signal that gravel layers exist in the area, then, in my opinion, this represents the worst-case scenario. If gravels are encountered in minor volumes during investigations/construction, excavation and removal of the gravels will be required. If gravels are encountered across significant lengths of the section, excavation and removal cannot be used and then appropriate retaining solutions will need to be designed and built. It should be noted that based on Section 4.1 and Appendix 2, Taupo Pumice Formation and Oruanui Formation at the site are not anticipated to contain significant amounts of gravel.

9.17. The bridge location is likely to be underlain by some recent as well as Holocene river deposits which are underlain by Taupo Pumice Formation ignimbrite or Oruanui Formation ignimbrite. The bridge will be a single span with abutments clear of the 1% AEP stream flow. There will be no piers within the waterway. In a typical scenario, piles embedded in the Taupo Pumice Formation ignimbrite or Oruanui Formation ignimbrite, which have a high friction angle, can be used as a foundation solution for the abutments. The worst-case scenario will likely involve a very soft, compressible layer in the top few metres due to the proximity of the stream and the lake which is underlain by pumiceous material or ignimbrite. In this worst-case scenario, piles will have to be designed to account for negative skin friction in the top compressible layer. The piles will have to be embedded in the Taupo Pumice Formation ignimbrite or Oruanui Formation ignimbrite and will be designed accordingly.

9.18. It should be noted that exposed pumiceous soils are prone to erosion and fritter and the surface deteriorates rapidly if not protected. Hence, they should be stabilized at the earliest opportunity during and subsequent to the construction phase by minimising flow of water over pumiceous soils, and covering the pumiceous soils with topsoil and vegetating the exposed surfaces.

10. SUBMISSIONS

10.1. I have read the following submissions. Submissions related to geotechnical concerns and my comments are tabulated below.

Submitter	Submission Points	Comment
Waikato Regional Council ("WRC")	The proposed development area contains a potential erosion feature, the 'bowl' (see Figure 1 below) that does not appear to have been sufficiently addressed in the geotechnical reporting. Housing is proposed within close proximity to the 'bowl' feature. However, information provided by the applicant is not sufficient to confirm whether or not the bowl comprises a primary hazard zone and therefore an intolerable risk. WRPS Section 6A(h) directs new development away from natural hazards. In addition, WRPS Policy 13.1(c)states that the creation of new intolerable risk is to be avoided. District Plans shall incorporate a risk-based approach into the management of subdivision, use and development in relation to natural hazards and shall ensure that new development is managed so that natural hazard risks do not exceed acceptable levels (Section 13.1.1(a)). An intolerable natural hazard risk is defined in the WRPS as 'risk which cannot be justified and risk reduction is essential e.g. residential housing being developed in a primary hazard zone'. A primary hazard zone is 'an area in which the risk to life, property or the environment from natural hazards is intolerable'.	See discussion in Section 9 and Appendix 12. Hence, in terms of the WRPS: • The land affected by the Whareroa North proposal (shown on the "Whareroa North Concept Plan" in proposed Taupo District Plan Appendix 8) does not constitute a *"primary hazard zone" (*being *'an area in which the risk to life, property or the environment from natural hazards is intolerable');* • The proposal will not create an "*intolerable risk*" which the WRC submission states is defined as "*'risk which cannot be justified and risk reduction is essential e.g. residential housing being developed in a primary hazard zone'.* Accordingly, I consider that the appropriate assessment has been undertaken in terms of the relevant parts of WRPS: • **Policy 13.1** that "*Natural hazard risks are managed using an integrated and holistic approach that:* *a) ensures the risk from natural hazards does not exceed an acceptable level;* *b) protects health and safety;*

Submitter	Submission Points	Comment
	The geotechnical reporting indicates the erosion and scouring (shown below) is being caused by underground processes. However, the investigation was limited to publicly available information and a surface inspection. The limitations of this approach and the possibility of other problems being present were noted by the proponents consultant. In a memorandum from Cheal Consultants Ltd (dated, 3 October 2018) to address items 3 and 4 in Taupo District Council's 15 February 2018 request for additional information it states: 'The area of erosion/scouring was inspected and evaluated. Essentially the scour is being caused by concentrated underground flow triggering slow erosion of a layer of loosely packed quartzitic/lithic sand present at the base of the scour process, leading to the posterior collapse of soil layers above, including the surface layer. The surface layer is not eroding purely as a result of surface run-off. The problem starts deeper down and leads to the surface issue.' The memorandum was replaced a fortnight later by a report from Cheal Consultants Ltd dated 18 October 2018. The report does not expand upon the information provided in the earlier memorandum or clarify the possible role of underground flow in the formation of the bowl. It offers a possible formation mechanism, but cautions about the information gathering process and the possibility of the existence of special conditions that have not been identified. The report states: 'The investigation carried out by MTM [in 2006] is considered a basic level investigation/analysis that did not allow them to directly discard that a landslide had	c) *avoids the creation of any new intolerable risk.*" • **Policy 13.2** that "*Subdivision, use and development are managed to reduce the risks from natural hazards to an acceptable or tolerable level including by:* a) *ensuring risks are assessed for proposed activities in land subject to natural hazards;* b) *avoiding intolerable risk in any new use or development in areas subject to natural hazards.*"

Submitter	Submission Points	Comment
	occurred on the bowl-shaped area nor to verify the source of the sandy/gravely materials deposited underneath the ash material noticed via trenching. The report does not investigate the reason of the failure noticed at the lowest part of the bowl or the genesis of the bowl-shaped feature. Based on Cheal specific assessment, the bowl-shaped area noticed on the proposed development area and at two areas further to the west, could indicate ancient meanders of the Whareroa Stream created when the level of Lake Taupo was at a higher level than present. This could also be used to clarify the source of the sandy/gravelly materials noticed on the lower part of the bowl, which potentially were mobilised and deposited by the power of the stream flow.’ ‘As information over much of the site and surrounding land has been obtained solely from publicly available and provided information, and visual assessment of the land features there may be special conditions pertaining to this site which have not been identified by the undertaken analysis and which have not been taken into account in the report.’ The ‘bowl’ feature, and escarpment substrate may also have implications for the design of stormwater infrastructure required to service the proposed development. The stormwater management systems will need to be designed to ensure post-development hydrology remains as close to pre-development hydrology as possible. The stormwater management systems will also need to be designed to avoid or mitigate adverse effects on the receiving environment including the Whareroa Stream. Further information is therefore required, beyond the boundaries of the subdivision, to understand how the new development will meet the principles of WRPS 6A (e) and (h) to connect well	

Submitter	Submission Points	Comment
	with existing infrastructure and direct development away from hazard areas.	
Waikato Regional Council ("WRC")	WRPS Development Principles 6A(e) and (h) requires that new development connect well with existing and planned development and infrastructure and be directed away from natural hazard areas. Therefore, access to the proposed subdivision, and in particular, any constraints to access should form a key consideration in the plan change process. WRC submits that the practicalities of accessing the proposed development should be assessed through the plan change process so that the indicative route up the steep slope on the northern side of the Whareroa Stream can be given appropriate consideration.	See Section 9.17
Dr Ruth & Simon Ewen	Where the proposed road up to the subdivision is planned is likely unstable ground with evidence of many slips.	See Section 9.15
Ian Sutcliffe Michael Townson Miller	The proposal contains insufficient information concerning the potential geotechnical effects to confirm or otherwise that the land on which the proposal is intended is stable, and will not result in land slip or subsidence, which in turn will adversely implicate the ecology of the Whareroa Stream. Unstable geological area.	Refer to Section 9 of my evidence. Refer to Section 9 of my evidence.
Kia Paranihi	Outcomes of hapu hui have largely been worries about a raw scarp area above the stream which erodes at times of medium to heavy rainfall and also, the structure and placement of the bridge crossing of the stream. The scarp can erode, resulting in silt and pumice sand slipping into the stream and causing change to its outfall and nature. At	The submitter supports the plan change. For technical details, refer Section 9.

Submitter	Submission Points	Comment
	the lakeside crossing of the stream it is possible to cross it at ankle depth one day and above knee depth the next. This is a shock if you are unaware and there was a concern that the problem would increase with the development. While we acknowledge that this is a naturally and regularly occurring event every now and then given the pumice nature of the lakeside soil structure, we conveyed to the development consultants our wish to have this minimised to achieve stabilisation of the land as we are not far away. The developers response as outlined in the application is more than satisfactory and we are assured of ongoing consultation on the matter.	
Robert & Jo Colman	The land to which a bridge is proposed to be built upon is very unstable.	Refer to Section 9.16

11. GEOTECHNICAL SECTION OF THE SECTION 42A REPORT

11.1. I have read the geotechnical evidence presented as Attachment H of the Section 42A report. Geotechnical concerns identified in the geotechnical evidence and my response to them are discussed in Section 9 above and in Appendix 12. Some specific comments are set out below.

Reference	Submission Points	Comment
Section 8.5	Deep geotechnical investigation such as additional CPTs (with locations accurately recorded) and/or machine drilled boreholes, are recommended to support proposed plan change applications as per New Zealand Geotechnical Society (NZGS) and Ministry of Business, Innovation and Employment (MBIE) Earthquake Geotechnical Engineering Practice Guidelines. The Guidelines are draft, and it is not mandatory to follow the guidelines, however they are widely accepted as 'best practice' in the geotechnical engineering industry. I	See Section 9.2

Reference	Submission Points	Comment
	refer to Table 2.1 in Module 2 of the guidelines, which recommends a minimum of five deep site investigation locations at the plan change stage, for a site with an area greater than 1.0 hectare.	
Section 9.4	Many of the geo-hazards tabled in the proponent's email are significant with problematic, complex and costly engineering solutions to mitigate the effects of the geo-hazard. Assessment of these geo-hazards has not been provided by the Proponent.	In my professional opinion all of these geo-hazards are routinely encountered in and around Taupo as well as near Rotorua. Engineering solutions, even though complex and expensive, exist to mitigate effects of these geo-hazards and are used routinely by professional engineers. I also consider that expenses related to developing the subdivision including investing in geotechnical solutions is the developer's prerogative.
Section 10.3	The volume of earthworks required to remediate the site depends entirely on the extent and thickness of the identified compressible soil layer – which as identified above there is no definitive information on which provided by the Proponent. Balancing the volume of earthworks, with retaining walls and drainage improvements will also need to be considered.	I agree that some earthworks will be required at the site. To calculate volumes and balance of earthworks required, all engineering investigations, analysis and design activities need to be completed. This is usually performed over the entire consenting process and is not considered necessary at the Plan Change stage. I consider it more important to understand that these geo-hazards are routinely encountered in and around Taupo as well as near Rotorua and engineering solutions exist to mitigate effects of these geo-hazards and are used routinely by professional engineers.
Section 10.12	The disposal of stormwater on-site has significant geotechnical consequences if not adequately managed. The feasibility of capturing and disposing stormwater run-off, from both road reserves and future dwellings, via a piped network which outlets to the Whareroa Stream	The possibility of piping the stormwater to the Whareroa stream to provide outlets for the ponds was discussed by Cheal's civil engineer (Tony Kelly) and also geotechnical engineer (Andres Martinez). It was not pursued due to the possible impacts on the stream including water

Reference	Submission Points	Comment
	has not yet been explored, however would be a relatively orthodox (although much costlier) alternative if disposal to ground is found to be unsuitable.	quality degradation and erosion as well as the probability of water flowing along the outside of the pipe and likely causing erosion. This approach is also contrary to current best industry practice and cultural expectations.
Section 10.13	In addition to the considerations above, the design life of the proposed stormwater ponds will need to be agreed with TDC, such that they understand when the ponds will need refurbishment or replacement. Depending on the materials utilised, a finite life is assumed. The construction of the ponds would need to be very closely monitored and a maintenance and monitoring programme agreed with TDC, as any leakage or failure of the ponds would almost certainly result in catastrophic damage to the steep land adjoining the proposed development.	The design life can be discussed and finalised during the subdivision consent stage. It is noted that section 63 of Mr Stokes' evidence states: - "*It's not my intention to examine the technical details of the proposed solutions in detail in this evidence. The Council has control during the subdivision resource consent process of the specific design of the infrastructure, and this application doesn't extend into the engineering detail, as would be expected*".
Section 11.3	I do not believe it is appropriate to assume all geohazards can be investigated, assessed and mitigated through subdivision and building consent conditions.	If appropriate geotechnical investigations, laboratory testing, monitoring, engineering analyses and design is performed, all geo-hazards can be assessed and mitigated at any particular consenting stage or across multiple consenting stages. On the contrary, I do not think that geo-hazards can be investigated, assessed and mitigated appropriately based on the limited investigations described in Section 8.7 of Ms Phillips evidence as these would provide insufficient information. Hence, a draft outlining the comprehensive geotechnical investigations that are intended to be performed by the Applicant has been presented in Appendix 12. These investigations will enable us to assess and confirm all geo-hazards and to provide necessary recommendations.

Reference	Submission Points	Comment
Section 11.2	Due to the information gaps it is not possible to determine what the realistic geotechnical costs associated with developing the land under a Residential Environment would be. Therefore, based on 'worst-case' assumptions, the costs associated with geotechnical development of the land are likely to be significantly more than development of other greenfield sites of the same size not affected by similar geohazards.	Geo-hazard assessment has been presented in Section 9.2 of my evidence. Investigations that will be performed at the further investigation stage have been presented in Section 9.7 and Appendix 12 of my evidence. The most important conclusion that I draw based on my assessment that was performed while preparing this evidence is that all of these geo-hazards are routinely encountered in and around Taupo as well as near Rotorua. Engineering solutions, even though complex and expensive, exist to mitigate effects of these geo-hazards and are used routinely by professional engineers. As stated above, I also consider that expenses related to developing the subdivision including investing in geotechnical solutions is the developer's prerogative.
Attachment 1	Bearing Capacity	Information on the bearing capacity at the site is made during the building consent stage.
Attachment 3	PPC36 Whareroa – Geotech Issues Meeting Minutes (7 April 2020)	Point numbered 4.c. in Attachment 3 of Attachment H of the Section 42A report is incomplete. It was discussed and concluded during the meeting that "all geotechnical issues can likely be mitigated (but that such measures may not be economically viable)". That discussion and conclusion was not included by the Taupo District Council in the minutes that are presented as Attachment 3 of Attachment H of the Section 42A report despite our requesting Taupo District Council to include that in the minutes.

Reference	Submission Points	Comment
		Our e-mail and Taupo District Council's response has been presented in Appendix 14.

12. TECHNICAL CONCLUSION

12.1. While there are geo-hazards like instability, liquefaction susceptibility, lateral spreading, flow liquefaction, compressible soils, settlement/ subsidence including differential settlements, piping/ underground erosion, effects and/ or appropriateness of onsite soakage, effects on the "bowl" and the scar that can potentially affect the site, all of these geo-hazards are routinely encountered in and around Taupo as well as near Rotorua. Engineering solutions exist to mitigate effects of these geo-hazards and are used regularly by professional engineers. The Table copied below summarises typical and worst-case scenario and related mitigation solutions as discussed in Section 9. Hence, even if the worst-case scenario is encountered, I consider, that the site and access corridor is or can be made to be suitable for residential development from a geotechnical perspective.

	Typical Scenario	Worst Case Scenario
Foundation for the bridge	The bridge location is likely to be underlain by some recent as well as Holocene river deposits which are underlain by Taupo Pumice Formation ignimbrite or Oruanui Formation ignimbrite. The bridge will be a single span with abutments clear of the 1% AEP stream flow. There will be no piers within the waterway. In a typical scenario, piles embedded in the Taupo Pumice Formation ignimbrite or Oruanui Formation ignimbrite, which have a high friction angle, can be used as a foundation solution for the abutments.	The worst-case scenario will likely involve a very soft, compressible layer in the top few metres due to the proximity of the stream and the lake which is underlain by pumiceous material or ignimbrite. In this worst-case scenario, piles will have to be designed to account for negative skin friction in the top compressible layer. The piles will have to be embedded in the Taupo Pumice Formation ignimbrite or Oruanui Formation ignimbrite and will be designed accordingly.
Cuts along the access road	I consider that the general stratigraphy through the area where the access road is proposed will be colluvium underlain by silt/sand. These upper layers are susceptible to shallow slope failures (as seen in Appendix 9) when they are not held together by grass or tree roots and/or during rainfall events. I consider that "unstable ground", "land slip" identified	If the machine drilled boreholes along the access corridor (discussed in Section 9) signal that gravel layers exist in the area, then, in my opinion, this represents the worst-case scenario. If gravels are encountered in minor volumes during investigations/construction, excavation and removal of the gravels will be

	by submitters is likely to be these shallow slope failures. These will be underlain either by Taupo Pumice Formation ignimbrite and Oruanui Formation ignimbrite. Based on my experience of pumiceous soils in and around Taupo, I consider that pumiceous soils/ignimbrite can stand up very well at slopes (batters) of 1:4 (1 horizontal to 4 vertical) (refer Appendix 10). If the cut heights are greater than 5m, then benching (terraces), with a minimum width of 3m, should be utilised.	required. If gravels are encountered across significant lengths of the section, excavation and removal cannot be used and then appropriate retaining solutions will need to be designed and built.
Surface Runoff	Stormwater from a 1% AEP event will be conveyed via the road reserve to an attenuation pond which will discharge from the pond via sheet flow that will be restricted to the pre-development 10% AEP flow rate well away from the scar as mentioned in Ref. 1.6.5. Additionally, surface runoff has a low potential to incise the scar as per Ref. 1.6.4. Based on high friction angles of pumiceous material and no major instabilities being observed away from the scar, it is considered that the ground between the pond and the bank will be stable and be able to accommodate periodic sheet flows from the pond. Hence, I consider that worsening of the scar due to surface runoff as well instability of the ground between the pond and the bank will not occur and this scenario is referred as the typical scenario. These details will be assessed as a part of geotechnical investigations and analysed before the subdivision consent application is submitted.	If the erosion risk is considered to be high even if sheet flow is restricted to the pre-development 10% AEP flow rate, the outflow swale lengths can be increased so that sheet flow occurs over a larger ground area. Alternatively, the ponds can be relocated and sheet flows can be diverted over areas which are assessed to be stable.
Drilled soak holes	There will not be any surface runoff through the scar as a part of the proposed development as opposed to no controls being in place to manage stormwater currently. The possibility of causing underground flow paths will be assessed during the resource/subdivision consent	If underground flow paths are encountered during the resource/subdivision consent stage, there should be no on-site soakage in the pre-developed Catchment B.

	stage. Hence, the development of the land is a beneficial scenario rather than a threat when compared to the current scenario.	

12.2. A draft outlining the geotechnical investigations that are intended to be performed is presented in Appendix 12. These investigations will enable us to assess and confirm all geo-hazards and to provide necessary recommendations to mitigate their effects. Hence, I recommend that all geotechnical investigations be performed as one campaign after the plan change request is approved and before a subdivision consent application is submitted.

Harshad Sham Phadnis

APPENDICES:

1. Verification of Geotechnical Constraints for Residential Development by Andres Martinez dated 18 October 2018. Ref. IBA 1070L150. Cheal Consultants Limited.
2. Geological Setting of the Area.
3. Information available on NZGD.
4. Report on the Geotechnical Aspects of the Whareroa Village Subdivision, Lake Taupo by S.J. Carryer dated 22 April 1986. Ref. 156/0764. Carryer & Associated Ltd.
5. Site Assessment and Supplementary Geotechnical Engineering Appraisal – Proposed Whareroa North Residential Subdivision – Hauhungaroa No. 6, Whareroa Road North, West Lake Taupo by M.T. Mitchell dated 19 October 2006. Ref. T – 9036/1. Mark T Mitchell Limited.
6. Preliminary Stormwater Assessment by T. Kelly dated 26 September 2019. Ref. IBA 1070 Rev. 4. Cheal Consultants Limited.
7. Cone Penetration Test Locations and Data.
8. The “bowl” at the site.
9. Shallow slip failure.
10. Examples of pumiceous soils standing at steep gradients.
11. Representation of cuts involved for the proposed access road
12. Proposed Geotechnical Investigations.
13. Proposed Plan Change 36 – Whareroa North – Initial Geotechnical Review by M. Phillips and I. Gray dated 31 March 2020. Ref. 2-37780.00. WSP Opus.
14. Cheal’s version of PPC36 Whareroa – Geotech Issues Meeting Minutes (7 April 2020).

Appendix 1: Verification of Geotechnical Constraints for Residential Development by Andres Martinez dated 18 October 2018. Ref. IBA 1070L150. Cheal Consultants Limited. (**Please see Appendix in ShareFile sub-folder of my evidence)**.

Appendix 2: Geological Setting of the Area

Geology 1:
Key name: Oruanui Formation ignimbrite (Taupo Group) of Taupo Volcanic Centre
Simple name: Late Pleistocene igneous rock
Main rock name: ignimbrite
Description: Non-welded ignimbrite and phreatomagmatic fall deposits, and reworked ignimbrite

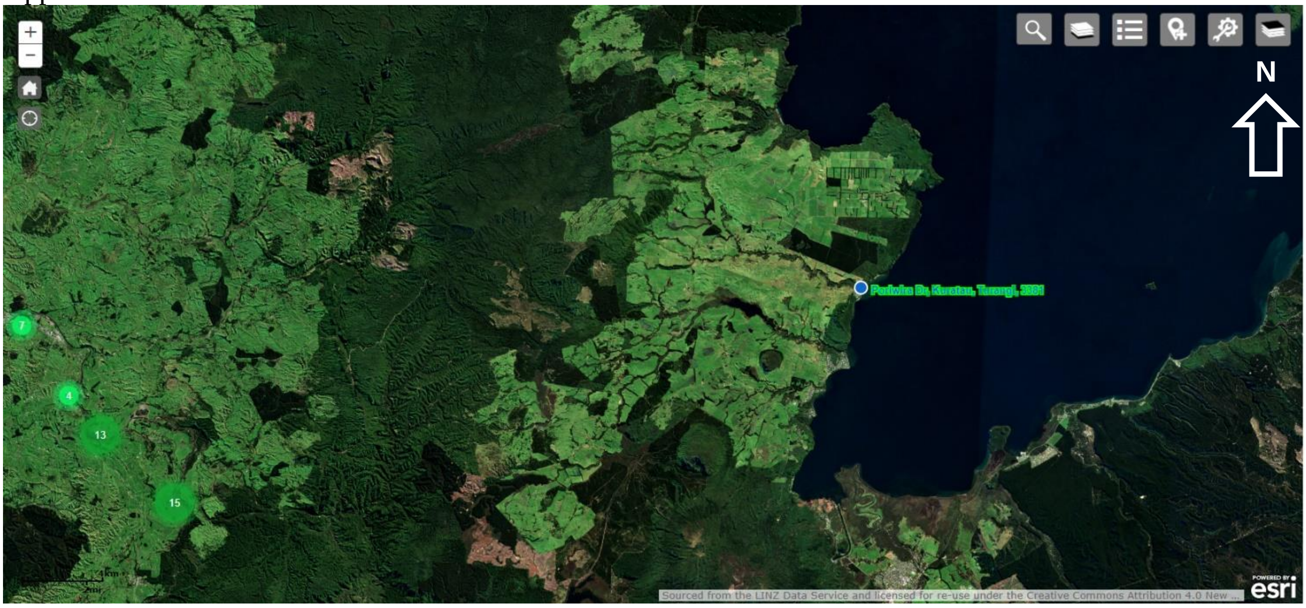
Geology 2:
Key name: Taupo Pumice Formation of Taupo Volcanic Centre
Simple name: Holocene igneous rock
Main rock name: ignimbrite
Description: Primary, non-welded ignimbrite and reworked deposits

Geology 3:
Key name: Holocene river deposits
Simple name: Holocene river deposits
Main rock name: pumice
Description: Predominantly pumice sand, silt and gravel alluvium with charcoal fragments derived from Taupo Pumice Formation

Geology 4:
Key name: Undifferentiated Taupo Group late Pleistocene rhyolite tephra of Taupo Volcanic Centre
Simple name: Late Pleistocene igneous rock
Main rock name: tephra
Description: Rhyolite tephra

Geology 5:
Key name: Undifferentiated Taupo Group middle Pleistocene rhyolite of Taupo Volcanic Centre
Simple name: Middle Pleistocene igneous rock
Main rock name: rhyolite
Description: Rhyolite lava variably with lesser pumice and breccia as a carapace

Appendix 3: Information available on NZGD

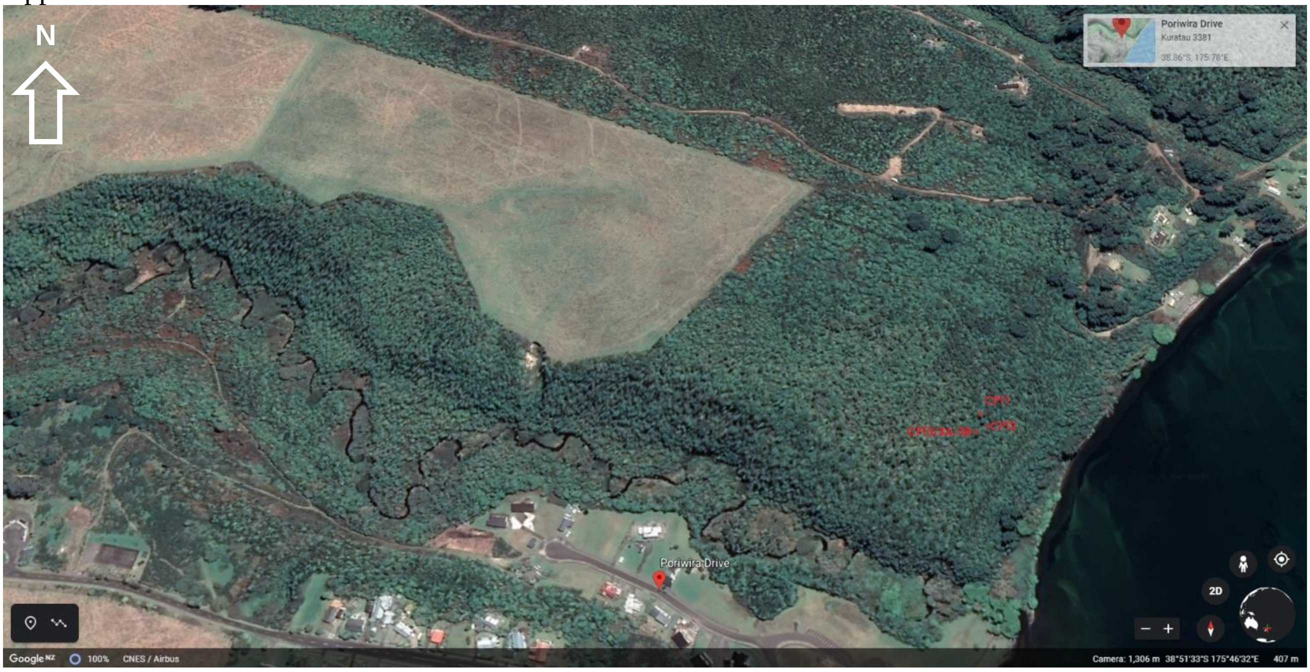


Appendix 4: Report on the Geotechnical Aspects of the Whareroa Village Subdivision, Lake Taupo by S.J. Carryer dated 22 April 1986. Ref. 156/0764. Carryer & Associated Ltd. (**Please see Appendix in ShareFile sub-folder of my evidence)**.

Appendix 5: Site Assessment and Supplementary Geotechnical Engineering Appraisal – Proposed Whareroa North Residential Subdivision – Hauhungaroa No. 6, Whareroa Road North, West Lake Taupo by M.T. Mitchell dated 19 October 2006. Ref. T – 9036/1. Mark T Mitchell Limited. (**Please see Appendix in ShareFile sub-folder of my evidence)**.

Appendix 6: Preliminary Stormwater Assessment by T. Kelly dated 26 September 2019. Ref. IBA 1070 Rev. 4. Cheal Consultants Limited. (**Please see Appendix in ShareFile sub-folder of my evidence)**.

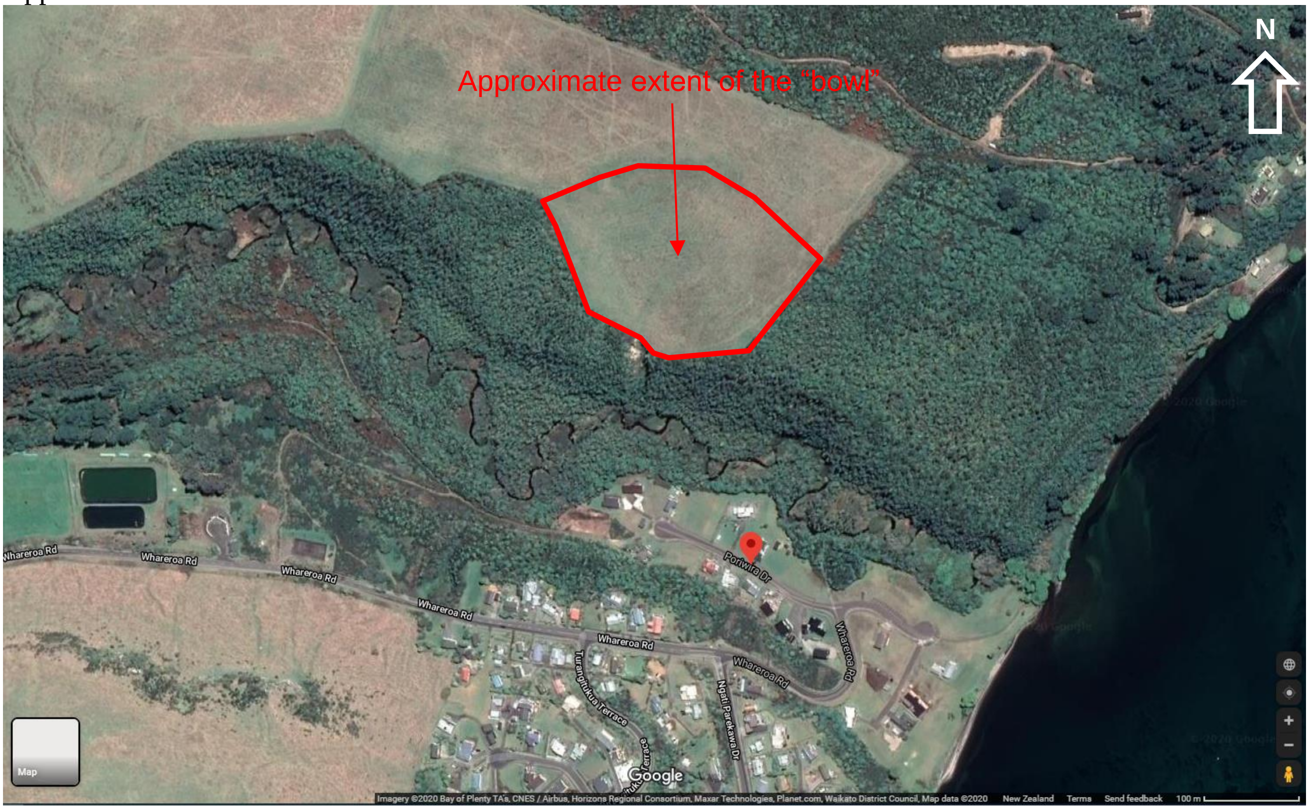
Appendix 7: Cone Penetration Test Locations and Data



Test	Reported Coordinates – Assumed to be as per the New Zealand Geodetic Datum 1949 (NZGD49)		Coordinates Interpreted as per the World Geodetic System 1984	
CPT1	- E2751534	N6256998	-38.857357	175.782581
CPT2	- E2751535	N6256988	-38.857446	175.782596
CPT3/3A/3B	- E2751525	N6256979	-38.857531	175.782484

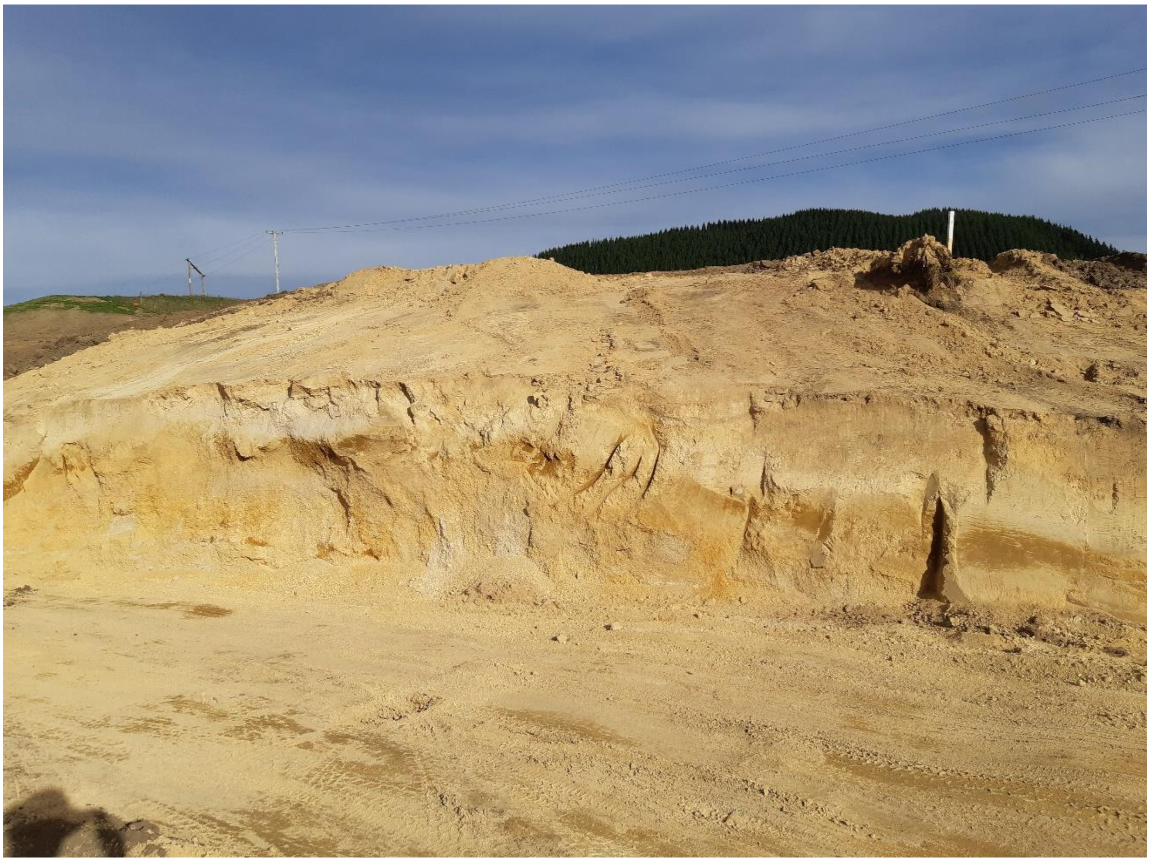
(Please see attachments to this Appendix in the Sharefile sub-folder of my evidence).

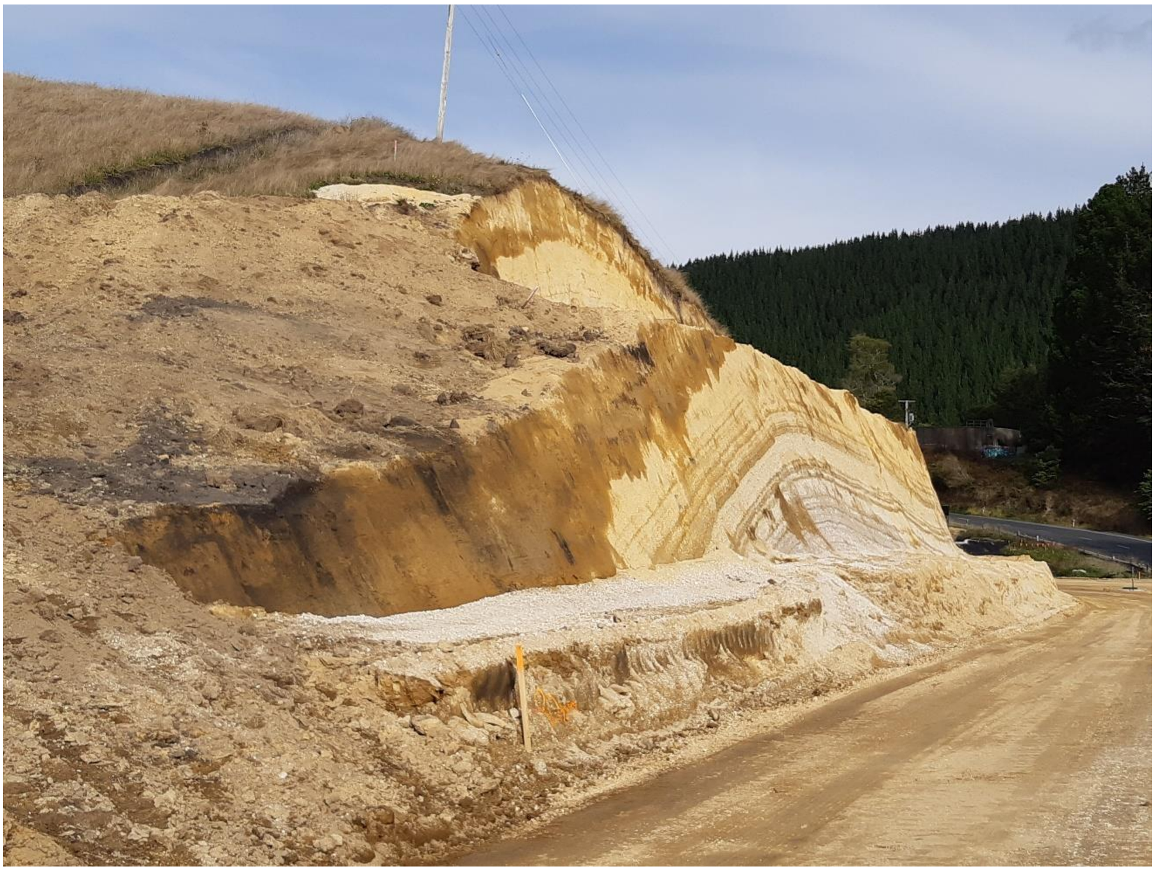
Appendix 8: The “bowl” at the site



Appendix 9: Shallow Slip Failure

Appendix 10: Examples of pumiceous soils standing at steep gradients





Appendix 11: Representation of cuts involved for the proposed access road
(**Please see Appendix in ShareFile sub-folder of my evidence)**.

Appendix 12: Proposed Geotechnical Investigations

MH: Up to 13 numbers of machine-drilled borehole to RL 355m as a minimum or to refusal, whichever occurs earlier.
CPT: Up to 15 numbers of cone penetration test to RL 355m as a minimum or to refusal, whichever occurs earlier.
S: Up to 8 numbers of Scala test to estimate the California Bearing Ratio to 3m or refusal, whichever occurs earlier.
So: Up to 7 numbers of soakage tests.

MH6/CPT6 indicates that a machine-drilled borehole as well as a cone penetration test is proposed at that location.

Linear shrinkage tests, Atterberg limit tests, unconfined compressive tests, density tests, consolidation tests and particle size distribution tests will be performed on samples recovered from the machine-drilled boreholes.

(Please see attachments to this Appendix in the Sharefile sub-folder of my evidence).

Appendix 13: Proposed Plan Change 36 – Whareroa North – Initial Geotechnical Review by M. Phillips and I. Gray dated 31 March 2020. Ref. 2-37780.00. WSP Opus.
(**Please see Appendix in ShareFile sub-folder of my evidence)**.

Appendix 14: Cheal’s version of PPC36 Whareroa – Geotech Issues Meeting Minutes (7 April 2020). (**Please see Appendix in ShareFile sub-folder of my evidence)**.