# Pukekohe North Substation

## Geotechnical Investigation Report

Ergo Consulting Limited

**Reference: AKLGE284189**

31 August 2021

# PUKEKOHE NORTH SUBSTATION

## Geotechnical Investigation Report

**Report reference number: AKLGE284189**
31 August 2021

## PREPARED FOR

**Ergo Consulting Limited**
PO Box 9717 Newmarket 1149
Level 1, 106 Carlton Gore Rd
Newmarket 1023

## PREPARED BY

**Tetra Tech Coffey**
Level 4, 25 Teed Street, Newmarket
Auckland
1023 New Zealand
p: +64 9 379 9463

NZBN 9429033691923

## QUALITY INFORMATION

### Revision history

| Revision | Description | Date | Author | Reviewer | Approver |
|---|---|---|---|---|---|
| v1 | GIR | 31/08/2021 | EP | CAA | LB |

### Distribution

| Report Status | No. of copies | Format | Distributed to | Date |
|---|---|---|---|---|
| v1 | 1 | **PDF** | **Steve Gaskin** | 24/08/2021 |

# CONTENTS

# LIST OF TABLES



# APPENDICES

# 1. INTRODUCTION

Ergo Consulting Limited has engaged Tetra Tech Coffey (NZ) Ltd (Tetra Tech Coffey) to undertake a geotechnical investigation for the proposed Counties Energy Pukekohe North Substation at 8 Whangapouri Road, Karaka. This geotechnical investigation report (GIR) has been prepared for the planned substation development in accordance with instructions received from Ergo Consulting Limited as per our proposal, dated 3 February 2021, reference 773-AKLGE284189. We understand that the proposed development will include earthworks with up to 1m cut and fill, retaining walls, three outdoor transformers, a 20m long switch room with a 2m deep basement, outdoor electrical equipment on steel stand and pad foundations and power pole structures. As a part of this development, the house currently located on the site will be relocated to the south-eastern portion of the property.

# 2. SITE INFORMATION

## 2.1 SITE DESCRIPTION

The site is located at 8 Whangapouri Road, Karaka. The site is relatively flat overall, with lower elevation areas and drainage channels to the north and east. The site is bound by Karaka Road to the South, Whangapouri Road to the west and rural farmland to the north and east. The rural property to the north houses one residential property. The site contains two residential properties located on the northern and eastern portions of the site with a driveway running west to east through the site’s centre. The site also houses a greenhouse and a chicken coop in the south-eastern corner.

## 2.2 HISTORICAL AERIAL PHOTOGRAPHS

Historical aerial photographs and satellite imagery of the site and surrounding area taken between 1996 and 2017 were obtained using Auckland Council GeoMaps website. Images were reviewed to assess the historic land use and to identify areas of interest to target in our ground investigation.

**Table 1: Selected Aerial Photograph Review**

| Year | Key Features |
|---|---|
| 1996 | The site is occupied by farmland. |
| 2001 | One residential property has been constructed on the eastern section of the site. |
| 2010/2011 | Another residential property has been constructed near the middle of the northern boundary of the site. A greenhouse has been constructed in the south east of the site. |
| 2017 | Extensions have been made to both existing houses and more vegetation has been planted around the central house. |

## 2.3 REGIONAL GEOLOGY

The Institute of Geological and Nuclear Sciences Geology Web map (https://data.gns.cri.nz/geology/) indicates that the site is underlain by Puketoka Formation soils, of the Tauranga Group sedimentary lithology (Upper Tertiary to Quaternary in age). The Puketoka Formation overlies East Coast Bays Formation sandstones and mudstones.

Commonly found materials in the Puketoka Formation include include pumiceous muds, sands, and gravels with muddy peats and lignite, rhyolitic pumice including non-welded ignimbrite, tephra and alluvial pumice deposits and massive micaceous sands.

The Institute of Geological and Nuclear Sciences Geology Web map also indicates Auckland Volcanic Field (AVF) deposits within 2km of the site. Our experience also suggests that AVF deposits are expected to overly Puketoka Formation soils in the area.

Commonly found materials in the AVF include basalt lava, scoria cones, volcanic breccia, ash, lapilli, and lithic tuff.

# 3. PROPOSED DEVELOPMENT

The proposed Counties Energy Pukekohe North Substation development is shown on the site plan and earthworks plan in Appendix A. Ergo Consulting Limited has indicated that the development proposed for Counties Energy at the site comprises earthworks with up to 1m of cut and fill, retaining walls, three outdoor transformers, a 20m long switch room with a 2m deep basement, outdoor electrical equipment on steel stand and pad foundations and power pole structures. The house currently located near the centre of the northern boundary will be relocated to the south eastern section of the property where the current greenhouse and chicken coop are located.

# 4. FIELDWORK

## 4.1 NEARBY INVESTIGATIONS

Nearby investigation data was sourced using the New Zealand geotechnical database (NZGD). Three water bores were drilled nearby as presented in Figure 1. Material encountered up to 25 metres was similar to that encountered during our investigation. The water bores also reached a depth where East Coast Bays Formation (ECBF) sandstones and mudstones were encountered. Water bore 78951, 81033, and 81903 encountered inferred ECBF rock at 24m, 50m, and 66m respectively, as shown in Figure 1.

**Figure 1: Location of nearby NZGD water bore logs.**

## 4.2 SITE SPECIFIC INVESTIGATIONS

Our site-specific fieldwork was carried out from July to August 2021 and comprised the following:

- Seven hand-auger boreholes (HA01-07) to depths of up to 2m (except HA03 to 5m), with shear vane measurements in fine grained soils.
- Four piezocone penetration tests (CPT01-04), to 20m depth.
- One machine borehole (BH01) to a depth of 24.95m with standard penetration tests (SPTs) every 1.5 metres.

Prodrill carried out the machine borehole drilling using a Fraste SLG rig and CPT testing using a Geoprobe CPT rig under observation by Tetra Tech Coffey.

Test locations are presented in Appendix A on the site plan. The machine borehole log, hand-auger logs and CPT results are presented in appendix B.

# 5. GROUND CONDITIONS

A summary geological model of the overall site is provided in Table 2.

**Table 2: Geological Model Across the Site**

| Unit | Description | Approximate Depth to Base of Unit |
|---|---|---|
| Topsoil | | 0.2m |
| Auckland Volcanic Field | Very stiff to hard silty clay and clayey silt | 5.0m |
| | Stiff to very stiff sandy silt. | 12.0m |
| Puketoka Formation | Stiff to very stiff sandy silt. | >18.5m |
| | Medium dense silty sand. | >25m |
| East Coast Bays Formation rock | Interbedded sandstones and siltstones. | Expected between 25-50m depth. |

## 5.1 TOPSOIL

Topsoil was encountered in all boreholes to a depth of 0.2 to 0.3m.

## 5.2 AUCKLAND VOLCANIC FIELD

Auckland Volcanic Field soils were encountered in all boreholes, up to a depth of up to 12.7m. The Auckland Volcanic Field soils comprised stiff to hard, orange, brown to pale grey silty clays, clayey silts, and sandy silts. This material had measured undrained shear strengths ranging from 80kPa to in excess of 216kPa, and typically averaging approximately 150kPa. SPT results ranged from an N-value of 3 to 14, typically averaging an N-value of approximately 8. CPT results found cone resistance (qt) in the Auckland Volcanic Field soils to range from 0.5MPa to 6.0MPa, typically averaging 2.0MPa. The clay content in Auckland Volcanic Field soils is sensitive to moisture content and trafficking.

## 5.3 PUKETOKA FORMATION

Puketoka Formation soils were encountered in the machine borehole underlying Auckland Volcanic Field soils. The base of the Puketoka Formation soils was not proven by our machine borehole investigation. This material generally comprised stiff to very stiff pale grey mottled orange sandy silts and silty clays. At depth Puketoka Formation soils comprised dense pale grey mottled orange silty sands. Shear vane measurements were unable to penetrate Puketoka Formation soils in the machine borehole. SPT results in the Puketoka Formation ranged from an N-value of 3 to 20, typically averaging an N-value of 11. CPT results found cone resistance (qt) in the Puketoka Formation to range from 1.5MPa to 10MPa, typically averaging 3.0MPa.

## 5.4 GROUNDWATER

Groundwater was observed in all HA boreholes at depths (with the exception of HA06) between 0.2m and 1.0m. The groundwater level in the machine borehole was unable to be observed due to the use of water as the drilling fluid. The summary of groundwater depths is displayed in table 3.

**Table 3: Groundwater Summary – Measured at Time of Investigation**

| Borehole No. | Groundwater Depth (m) |
|---|---|
| HA01 | 0.8m |
| HA02 | 0.8m |
| HA03 | 0.2m |
| HA04 | 0.9m |
| HA05 | 1.0m |
| HA06 | Not Encountered |
| HA07 | 0.8m |

During our site investigation ground water levels were encountered in our hand augers at shallow depths. As there is a proposed 2m deep foundation as part of the development, we recommended the installation of a piezometer standpipe to monitor groundwater depths to confirm the depth of groundwater for design. The 5m deep piezometer was installed on 28 July 2021. Groundwater measurements have been placed on hold due to COVID level 4 requirements and further groundwater monitoring will be carried out after Level 4 lockdown.

## 5.5 SEISMIC SITE SUBSOIL CLASS

Based on the nearby water bore logs sourced from NZGD, which show rock at a depth of 25m to 50m, the seismic site subsoil class is estimated to be D (Deep or soft soil), in accordance with NZS 1170.5 section 3.1.1. This is based on shear wave velocities correlated from the CPT results.

# 6. DESIGN CONSIDERATIONS

Based on our site-specific investigations, we consider the primary design considerations for the proposed development are the potential shallow groundwater depth and the sensitivity of the volcanic soils during construction. The soil profile comprises relatively stiff clayey soils, which have a low risk of static settlement. The relatively flat site has a low risk of slope instability and based on the geology and soil type, there is a very low risk of liquefaction.

## 6.1 GROUNDWATER

Based on the relatively shallow groundwater depths encountered during our investigations, consideration will need to be given to the impacts on potential dewatering for the switch room excavation, for potential trafficking damage during construction and the impacts on stormwater soakage design.

While groundwater was encountered at depths between 0.2m and 1.0m in 6 of the 7 HAs, the site investigation was carried out in winter, after a prolonged period of rainfall in Auckland. The soil profile is also relatively impermeable, meaning surface water takes a long time to disperse. We consider that the encountered groundwater level is likely to be perched and will be lower during dryer periods. We recommend construction is carried out over the dryer summer months.

Further recommendations will be supplied once additional groundwater measurements have been carried out.

## 6.2 CONSOLIDATION SETTLEMENT

All structural foundations must be designed to tolerate differential settlements of up to 1 in 240 (25mm over a 6m metre length) as required by the New Zealand Building Code Handbook, Appendix B Section B1/VM4, clause B1.0.2, under the serviceability limit state load combinations of NZS 1170.0, unless the structure is specifically designed to limit damage under a greater settlement.

While we consider the risk of static settlement to be low, we have carried out a consolidation settlement analysis using elastic soil theory. The analyses were done for all 4 CPTs, with assumed building loads of 35kPa for the switch room and 40kPa for the transformers and up to 1m of fill. The switch room is proposed to have a basement, which reduces the net load. Based on these foundation loads and the foundation dimensions and locations provided by Ergo, estimated SLS static consolidation settlements are considered to be negligible, and any differential settlements will be less the above building code requirements.

The excavation for the switch room basement will require an excavation approximately 2m deep. While this excavation may require some dewatering, due to the very stiff nature of the soil profile, we consider the risk of settlement of the surrounding ground as a result of potential dewatering, to be low.

## 6.3 DYNAMIC SETTLEMENT

As advised by the client, the substation and associated infrastructure are to be designed as Importance Level 4 (IL4) structures. Seismic design parameters have been calculated in accordance with NZGS (New Zealand Geotechnical Society) and MBIE (Ministry of Building, Innovation and Employment) guidelines, which recommend using the NZTA Bridge Manual. Earthquake design parameters are presented below in Table 4.

**Table 4: Earthquake Parameters for IL4 Structures, 50-year Design Life, Subsoil Class D**

| Seismic Scenario | Return Period | Earthquake Magnitude for Auckland | PGA |
|---|---|---|---|
| **SLS1** | 1/25yrs | 5.9 | 0.04g |
| **SLS2** | 1/500yrs | 5.9 | 0.15g |
| **ULS** | 1/2500yrs | 5.9 | 0.28g |

SLS – Serviceability limit state.

ULS – Ultimate limit state - Note this is considered higher that the NZTA lower bound ULS earthquake of M6.5, 0.19g.

PGA – Peak ground acceleration.

## 6.4 LIQUEFACTION ASSESSMENT

Liquefaction is a phenomenon that affects saturated, loose to medium dense sands and non-plastic silts as a result of strong ground shaking (i.e., earthquakes). It can lead to an almost total loss of strength within affected layers and can also cause vertical ground settlements and horizontal movement referred to as ‘lateral spreading’. While fine grained soils such as clays and plastic silts do not ‘liquefy’ in the classic sense, they may soften (lose strength) as a result of intense shaking. Based on the results of our desktop study and site-specific investigation, we do not consider the soil profile at this site to be susceptible to dynamic consolidation or liquefaction.

# 7. FOUNDATION RECCOMENDATIONS

Based on the results of our investigations, we consider that the proposed substation buildings, transformers, and equipment foundation pads are suitable to be supported on shallow foundations. The primary design considerations for the proposed development are the potential shallow groundwater depth and the sensitivity of the volcanic soils during construction.

A geotechnical ultimate bearing capacity of 300kPa may be assumed for the design of the proposed foundations constructed within competent natural ground at a depth of 0.6m below the finished ground surface. Using the above bearing capacity values for relatively shallow foundations, along with the appropriate strength reduction factors will serve to limit any total and differential foundation settlements to within allowable structural tolerances. As per B1/VM4 the recommended strength reduction factor is 0.5 for static design and seismic design.

Recommended soil parameters in the upper 5m of the AVF soils, to be used for retaining wall and pole foundation design are presented in Table 5. For undrained analyses, Su should be used and for drained analyses, effective cohesion c' and phi (Ø') should be used.

**Table 5: Soil Properties**

| Geologic Unit | Cohesion, c' (kPa) | Friction Angle, Ø' (degrees) | Shear Modulus, Go (MPa) | Poisson's Ratio, μ | Shear Strength, Su (kPa) |
|---|---|---|---|---|---|
| **Auckland Volcanic Field** | 8 | 34 | 40 | 0.35 | 100 |

For the allowable lateral pressure for long-term loads again for pole foundations, we would recommend using an ultimate lateral capacity of 4Su from the ground surface, i.e an ultimate lateral capacity of 600kPa. We recommend using a FOS of 3, so the allowable load on the poles under lateral load should not exceed 200 kPa.

For the purpose of calculating secondary response spectra, foundation stiffness can be estimated using the soil parameters in Table 5.

Foundations for the relocated house can assume an ultimate geotechnical ultimate bearing capacity of 300kPa from a depth of 0.2m in natural soil. However, due to the presence of sensitive soils we recommend a minimum timber pile depth of 400mm, as per NZS:3604:2011.

# 8. TEMPORARY CUT BATTERS

Foundation excavations will require temporary earthworks cut batters up to 2m high. The buried, pre-cast wall panels will then need to be waterproofed on the outside. This work should be carried out under geotechnical supervision. If any instability is observed in cut batters Tetra Tech Coffey should be consulted immediately and any waterproofing work in the excavations should be stopped until Tetra Tech Coffey have provided additional advice.

We recommend that all temporary cut batters have maximum slopes of 1V:1H and are covered to keep them dry and protect them from erosion. All surface stormwater flow should be directed away from the cut slopes.

Temporary crane pads for construction on the existing site at ground level, should remain well clear of any basement or foundation excavations and should assume a geotechnical ultimate bearing capacity of 300kPa. If additional capacity is required, Tetra Tech Coffey can provide a more detailed assessment with crane loading and pad details.

# 9. PAVEMENT DESIGN INFORMATION

## 9.1 CALIFORNIA BEARING RATIO

No site based dynamic cone penetrometer (DCP) tests or laboratory California Bearing Ratio (CBR) tests have been carried out. However, given that the volcanic soils are moderately sensitive in places we recommend that a subgrade CBR of 6% be adopted for pavement design in this material. When stripping the existing topsoil any unsuitable materials e.g., foreign objects and wet or soft soils should be removed and backfilled with compacted GAP40 hardfill.

## 9.2 PAVEMENT PROTECTION DURING CONSTRUCTION

Due to the sensitivity of the shallow soil profile, we recommend that pavement subsurface drainage is installed as soon as the pavement area has been stripped, and that the entire area is overlain by a geofabric, (geogrid if required) and a minimum depth 300mm hardfill layer to help protect the pavement subgrade from vehicle damage during building works.

## 9.3 PAVEMENT DRAINAGE

Long term performance of the pavement could be improved by the inclusion of a network of subsurface drains around the perimeter and beneath the area of the proposed pavement, preferably at least 0.6m below finished pavement surface level to help control surface water seepage under the proposed sealed area.

# 10. STORMWATER CONTROLS

It is important that due care is paid to the design and construction of the planned stormwater disposal system. These systems should serve to collect all stormwater runoff from the roof, and paved areas, and should connect directly into an approved stormwater disposal area located well clear of the proposed buildings and paved surfaces.

Due to the potentially shallow groundwater depths and the low permeability of the soil profile, soakage pits are unlikely to be suitable. Stormwater management will need to be using a specifically designed stormwater storage area, pond or similar.

Given the relatively impermeable soil profile and the minimal development surface areas relative to the site area, we consider that the development is unlikely to have any significant increase in surface runoff.

# 11. PLAN REVIEW

We recommend we are given the opportunity to review the developed design and final working drawings for this project prior to building consent application to ensure that our recommendations relating to site works and foundation design have been interpreted as intended.

# 12. CONSTRUCTION OBSERVATIONS

It is important that we are given the opportunity of inspecting the site clearing and earthworks operations, and all building platform and foundation excavations prior to the placing of hardfill or steel and the pouring of concrete to ensure that the ground conditions encountered are as anticipated based on the findings of this report. If they are not, we would be on hand to recommend the most appropriate design and/or construction modifications.

Upon satisfactory completion of these aspects of the works we would then be in a position to issue the appropriate Producer Statement – Construction Review (PS4) to Council. We require at least 24 hours' notice for site inspections.

# 13. LIMITATIONS

This report has been prepared solely for the use of our client, Ergo Consulting Limited, their professional advisers and the relevant Territorial Authorities in relation to the specific project described herein. No liability is accepted in respect of its use for any other purpose or by any other person or entity. All future owners of this property should seek professional geotechnical advice to satisfy themselves as to its ongoing suitability for their intended use.

The opinions, recommendations and comments given in this report result from the application of normal methods of site investigation. As factual evidence has been obtained solely from boreholes and CPT data, which by their nature only provide information about a relatively small volume of the subsoils, there may be special conditions pertaining to this site which have not been disclosed by the investigation and which have not been considered in the report.

If variations in the subsoils or site conditions occur from those described or assumed to exist, then the matter should be referred back to us immediately. If you have queries or require further clarification regarding aspects of this report, please contact the undersigned.

For and on behalf of Tetra Tech Coffey

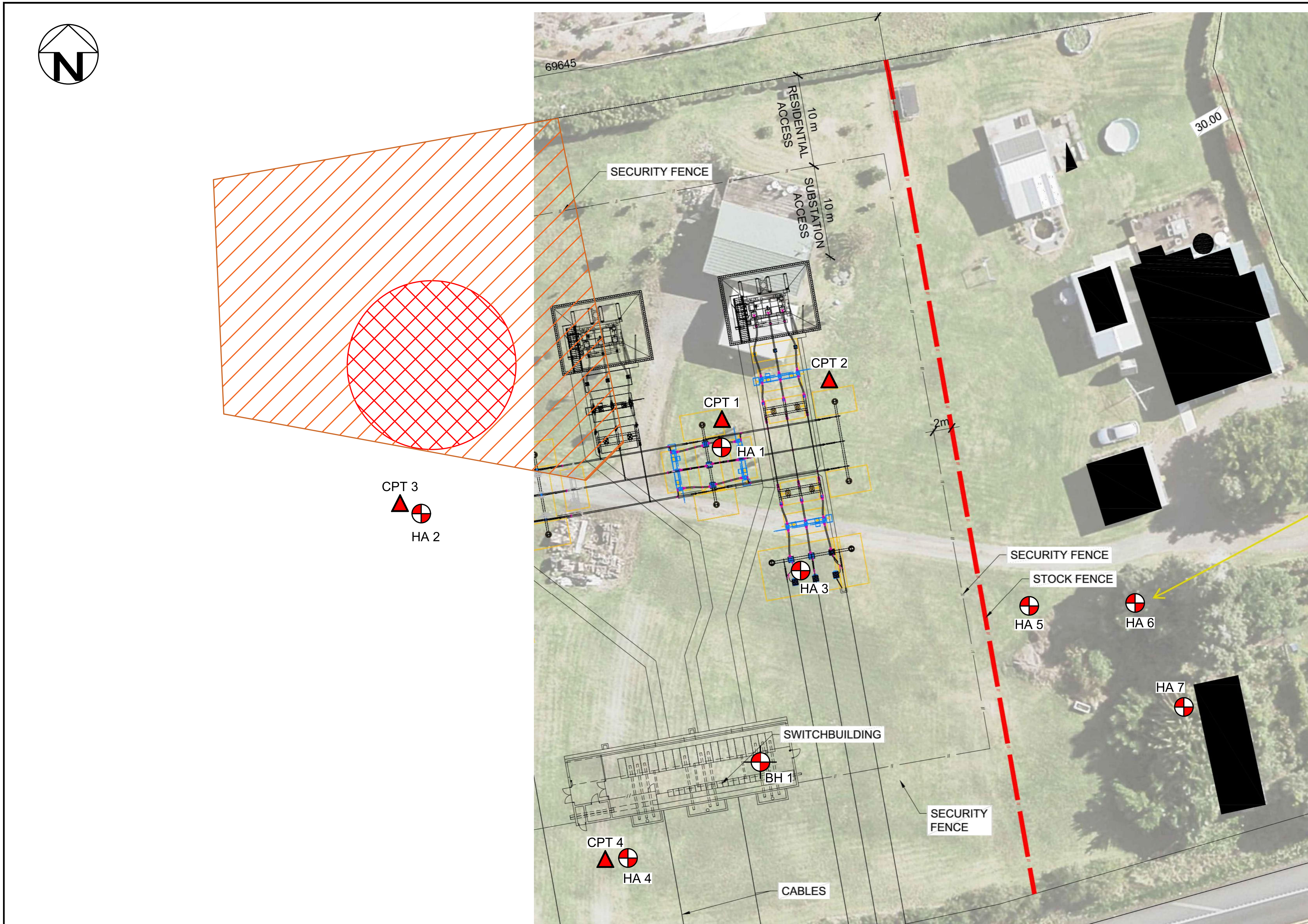
Prepared by

**Lee Buhagiar**
**BE(Hons) CPEng CMEngNZ Int PE (NZ)**
Associate Geotechnical Engineer

Reviewed and authorised by

**Chris Armstrong**
**CPEng CMEngNZ**
Associate Geotechnical Engineer

# APPENDIX A: SITE PLAN AND DEVELOPMENT PLAN




- PUKEKOHE NORTH SUBSTATION\7 DRAWINGS\CAD\773-AKLGE284189 - STANDARD\773-AKLGE284189.DWG

WHANGAPOURI ROAD
2
DP 473807
1
DP 160127
Dwelling
Garage
Shed
Deck
Pond
Pathway
Grass
SW