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PROPOSED REZONING OF LAND AT 259 ALFORD FOREST ROAD, ASHBURTON

GEOTECHNICAL INVESTIGATION REPORT

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PROPOSED REZONING OF LAND AT 259 ALFORD FOREST ROAD, ASHBURTON

GEOTECHNICAL INVESTIGATION REPORT

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SUMMARY

The purpose of the geotechnical investigation reported herein was to determine the subsoil conditions beneath the subject site as they may affect future residential development, with particular regard to foundation design considerations, and to determine the suitability of the subject site for the residential development, in support of a submission to generally rezone the area from Residential D Zone (Low Density) to Residential C Zone (Medium-Low Density).

It is understood that it is proposed to rezone the subject site, in the Ashburton District Plan, from Residential D Zone (Low Density) to Residential C Zone (Medium-Low Density).

The approximate location and extent of the subject site is shown on the appended Fraser Thomas Ltd drawing CH01763-G-01.

The subsoil information, presented in Appendix A of this report, indicates that the subject site is, in general, underlain by soils inferred to be alluvial sediments of Late Pleistocene age.

Based on the site appraisal and investigations, as reported herein, and on the basis of ground conditions existing at the time of the investigation reported herein, a "Recommended Building Line Limitation" has been determined for the site.

Foundation design recommendations are presented in Sections 10.0 and 11.0 of this report.

In general terms and within the limits of the investigation as outlined and reported herein, no unusual problems, from a geotechnical perspective, are anticipated with residential development at the subject site.

The site is, in general, considered suitable for its intended use, with satisfactory conditions for future residential building development, subject to the recommendations and qualifications reported herein, and provided the design and inspection of foundations are carried out as would be done under normal circumstances in accordance with the requirements of the relevant New Zealand Standard Codes of Practice.

Conclusions and recommendations are presented in Section 15.0 of this report.

PROPOSED REZONING OF LAND AT
259 ALFORD FOREST ROAD,
ASHBURTON

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TABLE OF CONTENTS

TABLES:

APPENDICES:

DRAWINGS:

PROPOSED REZONING OF LAND AT
259 ALFORD FOREST ROAD,
ASHBURTON

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation and appraisal undertaken for the proposed rezoning of the site located at 259 Alford Road, Ashburton. The subject site comprises eight parcels of land with a combined area of approximately 7,620 m^2.

It is understood that it is proposed to rezone the subject site, in the Ashburton District Plan, from Residential D Zone (Low Density) to Residential C Zone (Medium-Low Density).

The subject site is located on the south-western side of Alford Forest Road. Existing residential properties abut the north-western and south-eastern site boundaries. A rural property abouts the south-western site boundary.

The approximate location and extent of the subject site is shown on the appended Fraser Thomas Ltd drawing CH01763-G-01.

The subsurface conditions underlying the subject site have been investigated by means of seven machine excavated test pits, and associated Dynamic Cone Penetrometer (DCP) scala tests.

A visual appraisal of the site and a study of geological maps have also been undertaken.

The purpose of the geotechnical investigation reported herein was to determine the subsoil conditions beneath the subject site as they may affect future residential development, with particular regard to foundation design considerations, and to determine the suitability of the subject site for the residential development, in support of a submission to generally rezone the area from Residential D Zone (Low Density) to Residential C Zone (Medium-Low Density).

2.0 PREVIOUS REPORTS

A previous report titled "Earthquake Hazard Assessment," dated September 2002, was prepared by Geotech Consulting Ltd for the Environment Canterbury Regional Council. The September 2002 report was prepared in order to:

> *"…define and characterise the earthquake hazards in the Ashburton District."*

Figure 7.1, presented in the September 2002 report, indicates that the subject site is sited in 'Zone 1'. Zone 1 is defined as an area in which there is a "low potential" of liquefaction occurring.

3.0 SUMMARY OF 2010/2011 DAMAGING CANTERBURY EARTHQUAKE EVENTS

The Canterbury region has been subjected to significant seismic activity over the period September 2010 to June 2011 and beyond.

The significant damaging earthquake events are considered to be the following:

(a) 4 September 2010 (Moment Magnitude (M_w 7.1, epicentre depth = 11km),

(b) 22 February 2011 (M_w 6.2, epicentre depth = 5km),

(c) 13 June 2011 (M_w 6.0, epicentre depth = 6km),

(d) 23 December 2011 (M_w 5.9, epicentre depth = 6km).

The cyclic loading associated with these earthquake events has resulted in significant land deformation and associated building damage throughout some areas of the Canterbury region.

4.0 GEOLOGY

In carrying out the appraisal of the site, reference has been made to the New Zealand Geological Map 15, scale 1:250,000, Geology of the Aoraki Area.

This map indicates that the majority of the site is likely to be underlain by "*Light brownish grey river gravel, sand and silt, within abandoned outwash plains or low to mid-level terraces*" of Late Pleistocene age.

The map also indicates that the south-western part of the site is likely to be underlain by "*Grey river gravel, sand and silt associated with flood plains or low-level terraces*" of Holocene age.

The results of the machine excavated test pit investigation reported herein, in general, indicate that the surficial soils underlying the site, are likely to comprise alluvial sediments of Late Pleistocene age.

5.0 PROPOSED DEVELOPMENT

It is understood that it is proposed to rezone the subject site, in the Ashburton District Plan, from Residential D Zone (Low Density) to Residential C Zone (Medium-Low Density).

The approximate location and extent of the subject site is shown on the appended Fraser Thomas Ltd drawing CH01763-G-01.

6.0 FIELD INVESTIGATION

6.1 GENERAL

The field investigation comprised a visual appraisal, seven machine excavated test pits, and associated Dynamic Cone Penetrometer (DCP) scala tests.

The approximate locations of the investigation test positions are shown on Fraser Thomas Ltd drawing CH01763-G-01.

6.2 RESULTS OF VISUAL APPRAISAL

A visual appraisal of the subject site was undertaken by a Fraser Thomas Ltd engineering geologist on 3 August 2023.

The subject site is located on the south-western side of Alford Forest Road. Existing residential properties abut the north-western and south-eastern site boundaries. A rural property abouts the south-western site boundary.

The topography within the subject site is generally flat with some minor undulation.

The majority of site is vacant and vegetated with grass.

An existing single storey dwelling is located in the south-western corner of the site. Three detached structures, comprising a garage, and two sleepout type structures, are also located in south-western corner of the site.

The approximate inferred locations and extent of the existing structures are shown on drawing CH01763-G-01.

The crest of a slightly to moderately sloping terrace side slope abuts the south-western boundary of the subject site. The side slope generally slopes with a south-westerly aspect at slope angles of between approximately 14^{o} to the horizontal (1V: 4.011H) and 30^{o} to the horizontal (1V: 1.73H) for a vertical height of approximately 3.0 m.

No obvious signs of any deep-seated slope instability were observed for this terrace side slope.

The approximate inferred locations and extent of the existing site features are shown on drawing CH01763-G-01.

No obvious signs of any significant ground deformation, that could be attributed to liquefaction induced ground movement, were observed within the subject site, at the time of the investigation reported herein.

6.3 MACHINE EXCAVATED TEST PIT INVESTIGATION

Seven machine excavated test pits, numbered TP1 to TP7 inclusive, were put down at the site, in order to determine the nature and extent of the subsoils underlying the site.

The test pits were inspected and logged by a qualified Fraser Thomas engineering geologist.

The test pits were excavated to depths ranging between approximately 2.0 m and 4.0 m below the ground surface existing at the time of the investigation reported herein (i.e. the existing ground surface).

In situ undrained shear strength measurements were also carried out, where possible, using hand held shear vane equipment, within the cohesive soils encountered in the test pits.

The logs of Test Pits TP1 to TP7 inclusive are presented in Appendix A of this report.

Dynamic Cone Penetrometer (DCP) scala tests were carried out from the existing ground surface, and at depths ranging between approximately 0.9 m and 1.3 m below the existing ground surface, at the locations of the test pits, in order to determine the consistency of the cohesionless soils encountered in the test pits.

The results of the DCP scala tests are also presented in Appendix A of this report.

The approximate locations of Test Pits TP1 to TP7 inclusive are shown on drawing CH01763-G-01.

7.0 SUBSURFACE CONDITIONS

7.1 GENERAL

The subsoil information, presented in Appendix A of this report, indicates that the subject site is, in general, underlain by soils inferred to be alluvial sediments of Late Pleistocene age.

It has been assumed that even though the various subsoil strata (depths, thicknesses, and locations of groundwater levels) have been determined only at the locations and within the depths of the various test positions recorded herein, these various subsurface features can be projected between the various test positions. Even though such inference is made, no guarantee can be given as to the validity of this inference or of the nature and continuity of these various subsurface features.

7.2 TOPSOIL

A surficial layer of topsoil, generally comprising sandy silts, was encountered to depths ranging between approximately 0.3 m and 0.4 m below the existing ground surface, at the locations of the test pits.

7.3 ALLUVIAL SEDIMENTS

7.3.1 Sandy Silts

The results of the machine excavated test pit investigation, undertaken at the subject site, indicate that the surficial topsoil is generally underlain by a surficial layer of sandy silts, inferred to be alluvial sediments of Late Pleistocene age. These sediments were encountered to depths ranging between approximately 0.8 m and 1.8 m below the existing ground surface, at the locations of the test pits, corresponding to a layer thickness of between approximately 0.5 m and 1.5 m.

In situ undrained shear strength values of between approximately 83 kPa and greater than 200 kPa, were generally measured in the cohesive soils, using hand held shear vane equipment, corresponding to a stiff to hard consistency.

7.3.2 Sandy Silty Cobbly Gravels

The surficial cohesive soils at the site are generally underlain by a layer of material, generally comprising sandy silty cobbly gravels. These soils were generally encountered at depths ranging between approximately 0.8 m and 1.8 m below the existing ground surface, at the locations of the test positions. The gravel soils were encountered to the extent of the test pits.

The results of the DCP tests undertaken in the gravels, at the locations of the test positions, generally obtained DCP blow counts of between approximately 18 and greater than 50 blows per 100 mm penetration, corresponding to a SPT 'N' value of generally greater than 50, corresponding to a very dense consistency.

The logs of two existing water bores, presented in Appendix A of this report, put down approximately 15 m and 210 m respectively, to the north of the subject site, have also been sourced from Environment Canterbury records.

The existing water bore logs indicate that sandy gravels are generally located at shallow depths, which is consistent with the subsoil conditions encountered at the subject site. The bore logs indicate that these sandy gravels generally extend to depths in excess of approximately 35 m below the ground surface. Based on the foregoing, it is, in our opinion, likely that the gravel soils underlying the site extend to significant depths below the existing ground surface.

7.4 GROUNDWATER

Groundwater was not encountered during the investigations reported herein. However, based on information obtained from the existing water bore logs in the vicinity of the subject site, the groundwater level is inferred to be at a depth in excess of approximately 6 m below the existing ground surface, for analysis purposes.

8.0 LIQUEFACTION POTENTIAL ASSESSMENT

8.1 GENERAL

Liquefaction is defined as the phenomenon that occurs when soils are subject to a sudden loss in shear stiffness and strength associated with a reduction in effective stress due to cyclic loading (i.e. ground shaking associated with an earthquake).

The two main effects of liquefaction on soils are:

(a) Consolidation of the liquefied soils,

(b) Reduction in shear strength within the liquefied soils.

Liquefaction is considered to occur when the soils reach a condition of "zero effective stress". It is considered that only "sand like" soils can reach a condition of "zero effective stress" and therefore only "sand like" soils are considered to be liquefiable.

An indication that the underlying soils have been subject to liquefaction is the surface expression of ejected sand and water. This occurs as a result of the dissipation of excess pore water pressures generated within the liquefied soils as a result of the cyclic loading.

It should be noted that cohesive type materials or “clay like” soils are unlikely to be subject to liquefaction, as these soils (due to their nature) are unlikely to develop sufficient excess pore water pressures during cyclic loading to reach a condition of zero effective stress, i.e. the point of liquefaction.

However, “clay like” soils do develop some excess pore water pressures during cyclic loading which can result in consolidation settlement and a temporary reduction of the shear strength (i.e. softening) of the soils. Sensitive “clay like” soils are in particular susceptible to softening as a result of cyclic loading.

A liquefaction potential assessment has been undertaken for the soils underlying the subject site.

8.2 METHOD OF ANALYSIS

The New Zealand Geotechnical Society released Guidelines, in 2016, with the objective of summarising current best practice in earthquake geotechnical engineering with a focus on New Zealand conditions. The main purpose of the Guidelines is to promote consistency of approach to everyday engineering practice in New Zealand and, thus, improve geotechnical earthquake aspects of the performance of the built environment.

The Guidelines consists of six modules (identified as Modules 1 to 6 inclusive).

“Module 3: Identification Assessment and Mitigation of Liquefaction Hazards” of the Guidelines provides guidance on the identification of liquefaction hazards, and also provides details regarding different methodologies for determining theoretical liquefaction triggering.

The Module 3 guideline suggests a three-step process for the liquefaction assessment of sites, generally being:

(i) Step 1: Assessment of liquefaction susceptibility,

(ii) Step 2: Triggering of liquefaction,

(iii) Step 3: Consequences of liquefaction.

The Module 3 guideline refers to the methods suggested by “Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils”, dated October 2001. The guideline, among others, also refers to papers by Youd et al; Seed; Idriss; Boulanger; Robertson and Bray.

A liquefaction potential assessment of the soils underlying the subject site has been undertaken using the methods suggested by the Module 3 guideline.

8.3 ASSESSMENT OF LIQUEFACTION SUSCEPTIBILITY

The following soils are generally considered to be susceptible to liquefaction:

(a) Young (typically Holocene age) alluvial sediments (typically fluvial deposits laid down in a low energy environment) or man-made fills,

(b) Poorly consolidated/compacted sands and silty sands,

(c) Areas with a high groundwater level.

As discussed in Section 4.0 of this report, the geological map indicates that the site is likely to be underlain by "*Light brownish grey river gravel, sand and silt, within abandoned outwash plains or low to mid-level terraces*" of Late Pleistocene age.

As discussed in Section 7.3 of this report, the results of the machine excavated test pit investigation indicate the site is generally underlain by a surficial layer of stiff to very stiff cohesive soils, which are in turn underlain by very dense silty sandy gravels.

As discussed in Section 7.4 of this report, the groundwater level is inferred to be at a depth in excess of approximately 6 m below the existing ground surface, for analysis purposes.

Based on the foregoing, given the nature, age and consistency of the sediments underlying the subject site, i.e. a surficial layer of generally unsaturated stiff to very stiff cohesive soils, which are in turn underlain by very dense sandy gravels of Late Pleistocene age, it is our opinion that the soils underlying the site are unlikely to be susceptible to liquefaction in response to a future large earthquake event and that the risk of any significant liquefaction induced ground deformation occurring at the site, in response to a large earthquake event, is considered to be low.

It is therefore our opinion that the subject site, for foundation design purposes, should be assumed to be within Foundation Technical Category 1 (TC1), as defined by the MBIE guidance document, and that it is unlikely that liquefaction induced ground deformation could occur within the area in response to a large earthquake event, and that the ground settlements within the area in response to seismic loading should be considered to be "within normally accepted tolerances" as defined by the MBIE December 2012 guidance document.

It should also be noted that our assessment of the liquefaction susceptibility of the soils underlying the subject site is consistent with the assessment provided in the Geotech Consulting Ltd report, dated September 2002. As discussed in Section 2.0 of this report, Figure 7.1, presented in the September 2002 report, indicates that the subject site is sited in 'Zone 1', in which there is a "low potential" of liquefaction occurring.

9.0 LIMITATIONS ON BUILDING CONSTRUCTION

9.1 GENERAL

As discussed in Section 6.2 of this report, the crest of a slightly to moderately sloping terrace side slope abuts the south-western boundary of the subject site. The side slope generally slopes with a south-westerly aspect at slope angles of between approximately 14^o to the horizontal (1V: 4.011H) and 30^o to the horizontal (1V: 1.73H) for a vertical height of approximately 3.0 m.

No obvious signs of any deep-seated slope instability were observed for this terrace side slope.

The approximate inferred location and extent of the crest of the terrace side slope is shown on drawing CH01763-G-01.

Given the steepness of some parts of the side slope abutting the south-western site boundary, there is, in our opinion, a risk that this side slope may be subject to future shallow instability, which may adversely affect any proposed shallow foundations that are located in close proximity to the crest of the slope. In order to mitigate the risk of any instability of the existing terrace side slope in this area adversely affecting any future proposed shallow foundations, within the subject site, this section of the report provides the location of a "Recommended Building Line Limitation" (RBLL) for the site. The RBLL is set back from the crest of the terrace side slope.

9.2 RECOMMENDED BUILDING LINE LIMITATION

Based on the site appraisal and investigations, as reported herein, and on the basis of ground conditions existing at the time of the investigation reported herein, a “Recommended Building Line Limitation” has been determined for the site.

The "Recommended Building Line Limitation" shown in plan on drawing CH01763-G-01 represents, in our opinion, the limit up to which residential buildings can be constructed in accordance with the requirements of NZS 3604:2011, New Zealand Standard, Timber Framed Buildings.

The "Recommended Building Line Limitation" has generally been developed by applying a five metre setback from the crest of the terrace side slope.

The “Recommended Building Line Limitation” defines the boundary between:-

(a) A non-specific building foundation design zone, in which the foundations of any proposed residential building do not require specific design and which may, therefore, be constructed in accordance with the relevant New Zealand Standard Codes of Practice, providing the inspection and design of foundations are carried out as would be done under normal circumstances in accordance with the requirements of relevant New Zealand Standard Codes of Practice.

(b) A specific building foundation design zone, in which the foundations of any proposed residential building should be subject to specific design with particular regard to slope stability and settlement by a chartered professional engineer with the assistance of an engineer experienced in geotechnical engineering. Within this zone, the designer should, along with other criteria considered appropriate, undertake the following:

    (i) The design of a foundation system which properly takes into account the ground conditions at the specific location of any proposed structure.

    (ii) An assessment of founding depths and the locations of foundation lines to provide secure foundations for any proposed structure in the event of slope movement.

    (iii) The design of a foundation type to suit the proposed structure and to allow for soil creep and the distribution of lateral loads from the structure.

It should be noted that the “Recommended Building Line Limitation” shown in plan on drawing CH01763-G-01, is based on the existing ground surface profile.

It is recommended that any proposed building development be designed to satisfy the relevant requirements of the Building Code, so as to ensure compliance with the Building Act.

It should also be noted, based on the results of the investigation and appraisal reported herein, there is, in our opinion, a risk that land located within the specific foundation design zone determined for the site, may be subject to slope instability during or following heavy rainfall, which may result in the loss of land within the specific foundation design zone.

It is, however, our opinion, providing any proposed building development at the site located within the specific foundation design zone is subject to specific foundation design, as discussed in the foregoing Item (b), and is designed in accordance with the recommendations reported herein, that slope instability is unlikely to adversely affect future residential buildings at the site.

10.0 FOUNDATION DESIGN CONSIDERATIONS

10.1 GENERAL

It is our opinion that the soils underlying the subject site will exhibit only a low compressibility under the relatively light static foundation loads associated with a residential building development constructed in accordance with the requirements of NZS 3604: 2011, New Zealand Standard, Timber Framed Buildings.

It is, therefore, our opinion that settlement should not present a problem for future proposed residential development at the site, providing the inspection and design of foundations are carried out in accordance with the requirements of the relevant New Zealand Standard Codes of Practice, and in accordance with the recommendations presented in this report.

10.2 THE RISK OF THE PROPOSED DEVELOPMENT BEING ADVERSELY AFFECTED BY GROUND DEFORMATIONS ASSOCIATED WITH LIQUEFACTION

As discussed in Section 8.3 of this report, it is our opinion that the subject site, for foundation design purposes, should be assumed to be within Foundation Technical Category 1 (TC1), as defined by the MBIE guidance document, and that it is unlikely that liquefaction induced ground deformation could occur within the area in response to a large earthquake event, and that the ground settlements within the area in response to seismic loading should be considered to be “within normally accepted tolerances” as defined by the MBIE December 2012 guidance document.

Based on the foregoing, it is our opinion that an appropriate foundation solution for the site conditions would be a shallow foundation system designed in accordance with the requirements of NZS 3604: 2011, New Zealand Standard, Timber Framed Buildings (as modified by B1/AS1), founded in the underlying alluvial sediments.

An appropriately qualified and experienced Chartered Professional Engineer (CPEng), experienced in geotechnical engineering, should be engaged to inspect any foundation excavations, prior to the placement of any foundation materials, in order to confirm that the excavations are founded in competent alluvial sediments.

11.0 ALLOWABLE FOUNDATION BEARING PRESSURES

11.1 GENERAL

In this section of the report, ultimate bearing capacity values and strength reduction factors are provided in order to allow calculation of design (dependable) foundation bearing capacities, in accordance with the limit state design methods outlined in AS/NZS 1170: 2002, Structural Design Actions, by applying the appropriate strength reduction factors, as provided in this report, and the factored load combinations required by AS/NZS 1170. Allowable foundation bearing pressures are also provided, based on conventional factors of safety, for cases where unfactored load combinations are being considered.

11.2 SHALLOW PAD OR BEAM FOUNDATIONS

A minimum ultimate static bearing capacity value for vertical loading of 300 kPa is recommended for shallow concrete pads or beam foundations, founded in the underlying alluvial sediments. It is recommended that a strength reduction factor (Φ_{bc}) of 0.5 be adopted for limit state design in accordance with the requirements of AS/NZS 1170, resulting in a design (dependable) bearing capacity value of 150 kPa.

If unfactored load combinations are to be considered, the allowable foundation bearing pressures presented in Table 1 are recommended for shallow concrete pads or beam foundations, founded in the underlying alluvial sediments.

TABLE 1: ALLOWABLE FOUNDATION BEARING PRESSURES FOR SHALLOW CONCRETE PADS OR BEAM FOUNDATIONS FOUNDED IN THE UNDERLYING ALLUVIAL SEDIMENTS

Load Case	Factor of Safety	Allowable Bearing Pressure (kPa)
Dead Load and Permanent Live Load	3.0	100
Dead plus Live plus Transient Load	2.0	150

12.0 EXISTING SERVICE LINES

It is recommended that the location and depth of any buried services should be verified at the site prior to the commencement of foundation construction.

It is expected that any service line trenches would have been backfilled by conventionally acceptable means, which did not involve specific compaction. It would therefore be expected that some consolidation settlement of the service trench backfill could occur, which could result in lateral and vertical deformation of the undisturbed ground on each side of the trench backfill. The deformation is caused by the soil wedge behind the side wall of the trench moving downwards and inwards with time, towards the trench backfill as the backfill consolidates. The geometry of the soil wedge defines the theoretical zone of influence of the service trench backfill.

Due to the risk of consolidation settlement of the trench backfill occurring, it is recommended that, if any foundations of any proposed new building are located within the zone of influence of any existing service line, either the trench backfill be excavated and replaced with compacted hardfill or the foundations and floor of the proposed new building be designed to span across the trench backfill and the adjacent zone of influence.

The zone of influence is defined by a theoretical line projecting upwards in both directions from the centreline of the pipeline at the invert level of the pipeline at an angle of 45° to the vertical. The zone of influence is defined by the zone between the intersection point of the theoretical line and the ground surface on each side of the pipeline.

13.0 DEVELOPMENTAL EARTHWORKS

It is recommended that, unless the stability of any developmental earthworks (i.e. constructed for an access driveway, building platform or landscaping) is considered in detail by a chartered professional engineer experienced in geotechnical engineering, and particularly slope stability considerations, permanent fill end and cut slopes should be constructed to a maximum batter slope of 26° (1V:2H) with maximum batter heights of approximately 1.0 m. Any proposed higher permanent batter slopes should be subject to specific stability appreciation so as to determine stable limiting batter slopes.

It is recommended that any temporary excavated slopes be constructed to a maximum batter slope of 45^{o} (1V:1H), with a maximum batter height of approximately one meter. It is recommended that any temporary excavation slopes not be left unsupported for a period exceeding one month. It is also recommended that stormwater run-off be diverted away from the crest of any proposed temporary excavation slopes.

14.0 STORMWATER AND EFFLUENT DISPOSAL

It is understood that issues relating to stormwater discharge and effluent disposal will be addressed by others.

15.0 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations should be read together and not be taken in isolation.

15.1 CONCLUSIONS

Our conclusions based on the field data obtained from the site and as presented in this report, our visual appraisal of the site, our study of the geological maps relating to the area and our professional judgement and opinions, are as follows:

(a) In general terms and within the limits of the investigation as outlined and reported herein, no unusual problems, from a geotechnical perspective, are anticipated with residential development at the subject site.

The site is, in general, considered suitable for its intended use, with satisfactory conditions for future residential building development, subject to the recommendations and qualifications reported herein, and provided the design and inspection of foundations are carried out as would be done under normal circumstances in accordance with the requirements of the relevant New Zealand Standard Codes of Practice.

This report includes recommendations which will appropriately avoid, remedy or mitigate potential geotechnical hazards on the land subject to the application, in accordance with the provisions of Section 106 of the Resource Management Act.

In arriving at this conclusion and expressing this opinion, reliance has been based on the various topographical data as discussed herein and on subsoil information which has only been obtained at the locations and within the depths of the test positions reported herein. It has been assumed that this subsoil information can be projected between the various test positions. Even though such inference is made and forms the basis of the conclusions and opinions expressed herein, no guarantee can be given as to the validity of this inference or of the nature and continuity of the subsoils underlying the subject site.

(b) The purpose of the geotechnical investigation reported herein was to determine the subsoil conditions beneath the subject site as they may affect future residential development, with particular regard to foundation design considerations, and to determine the suitability of the subject site for the residential development, in support of a submission to generally rezone the area from Residential D Zone (Low Density) to Residential C Zone (Medium-Low Density).

(c) The results of the machine excavated test pit investigation, undertaken at the subject site, indicate that the surficial topsoil is generally underlain by a surficial layer of sandy silts, inferred to be alluvial sediments of Late Pleistocene age. These sediments were encountered to depths ranging between approximately 0.8 m and 1.8 m below the existing ground surface, at the locations of the test pits, corresponding to a layer thickness of between approximately 0.5 m and 1.5 m.

In situ undrained shear strength values of between approximately 83 kPa and greater than 200 kPa, were generally measured in the cohesive soils, using hand held shear vane equipment, corresponding to a stiff to hard consistency.

(d) The surficial cohesive soils at the site are generally underlain by a layer of material, generally comprising very dense sandy silty cobbly gravels. These soils were generally encountered at depths ranging between approximately 0.8 m and 1.8 m below the existing ground surface, at the locations of the test positions. The sandy silty gravels were encountered to the extent of the test pits.

The logs of two existing water bores, presented in Appendix A of this report, put down approximately 15 m and 210 m respectively, to the north of the subject site, have also been sourced from Environment Canterbury records.

The existing water bore logs indicate that sandy gravels are generally located at shallow depths, which is consistent with the subsoil conditions encountered at the subject site. The bore logs indicate that these sandy gravels generally extend to depths in excess of approximately 35 m below the ground surface. Based on the foregoing, it is, in our opinion, likely that the gravel soils underlying the site extend to significant depths below the existing ground surface.

(e) Groundwater was not encountered during the investigations reported herein. However, based on information obtained from the existing water bore logs in the vicinity of the subject site, the groundwater level is inferred to be at a depth in excess of approximately 6 m below the existing ground surface, for analysis purposes.

(f) Given the nature, age and consistency of the sediments underlying the subject site, i.e. a surficial layer of generally unsaturated stiff to very stiff cohesive soils, which are in turn underlain by very dense sandy gravels of Late Pleistocene age, it is our opinion that the soils underlying the site are unlikely to be susceptible to liquefaction in response to a future large earthquake event and that the risk of any significant liquefaction induced ground deformation occurring at the site, in response to a large earthquake event, is considered to be low.

It is therefore our opinion that the subject site, for foundation design purposes, should be assumed to be within Foundation Technical Category 1 (TC1), as defined by the MBIE guidance document, and that it is unlikely that liquefaction induced ground deformation could occur within the area in response to a large earthquake event, and that the ground settlements within the area in response to seismic loading should be considered to be “within normally accepted tolerances” as defined by the MBIE December 2012 guidance document.

(g) Based on the site appraisal and investigations, as reported herein, and on the basis of ground conditions existing at the time of the investigation reported herein, a “Recommended Building Line Limitation” has been determined for the site.

The "Recommended Building Line Limitation" shown in plan on drawing CH01763-G-01 represents, in our opinion, the limit up to which residential buildings can be constructed in accordance with the requirements of NZS 3604:2011, New Zealand Standard, Timber Framed Buildings.

(h) It is our opinion that the soils underlying the subject site will exhibit only a low compressibility under the relatively light static foundation loads associated with a residential building development constructed in accordance with the requirements of NZS 3604: 2011, New Zealand Standard, Timber Framed Buildings.

It is, therefore, our opinion that settlement should not present a problem for future proposed residential development at the site, providing the inspection and design of foundations are carried out in accordance with the requirements of the relevant New Zealand Standard Codes of Practice, and in accordance with the recommendations presented in this report.

(i) It is our opinion that an appropriate foundation solution for the site conditions would be a shallow foundation system designed in accordance with the requirements of NZS 3604: 2011, New Zealand Standard, Timber Framed Buildings (as modified by B1/AS1), founded in the underlying alluvial sediments.

15.2 RECOMMENDATIONS

Our recommendations based on the field data obtained from the site and as presented in this report, our visual appraisal of the site, our study of the geological maps relating to the area and our professional judgement and opinions, are as follows:

(a) It is recommended that any proposed shallow foundations be founded beneath the surficial topsoil into the underlying alluvial sediments.

(b) An appropriately qualified and experienced Chartered Professional Engineer (CPEng), experienced in geotechnical engineering, should be engaged to inspect any foundation excavations, prior to the placement of any foundation materials, in order to confirm that the excavations are founded in competent alluvial sediments.

(c) A minimum ultimate static bearing capacity value for vertical loading of 300 kPa is recommended for shallow concrete pads or beam foundations, founded in the underlying alluvial sediments. It is recommended that a strength reduction factor (Φ_{bc}) of 0.5 be adopted for limit state design in accordance with the requirements of AS/NZS 1170, resulting in a design (dependable) bearing capacity value of 150 kPa.

(d) It is recommended that the location and depth of any buried services should be verified at the site prior to the commencement of foundation construction.

Due to the risk of consolidation settlement of the trench backfill occurring, it is recommended that, if any foundations of any proposed new building are located within the zone of influence of any existing service line, either the trench backfill be excavated and replaced with compacted hardfill or the foundations and floor of the proposed new building be designed to span across the trench backfill and the adjacent zone of influence.

(e) It is recommended that, unless the stability of any developmental earthworks (i.e. constructed for an access driveway, building platform or landscaping) is considered in detail by a chartered professional engineer experienced in geotechnical engineering, and particularly slope stability considerations, permanent fill end and cut slopes should be constructed to a maximum batter slope of 26° (1V:2H) with maximum batter heights of approximately 1.0 m. Any proposed higher permanent batter slopes should be subject to specific stability appreciation so as to determine stable limiting batter slopes.

(f) It is recommended that any temporary excavated slopes be constructed to a maximum batter slope of 45° (1V:1H), with a maximum batter height of approximately one meter. It is recommended that any temporary excavation slopes not be left unsupported for a period exceeding one month. It is also recommended that stormwater run-off be diverted away from the crest of any proposed temporary excavation slopes.

16.0 LIMITATIONS

The professional opinion expressed herein has been prepared solely for, and is furnished to our client, Pajanti Ltd and Ashburton District Council for their purposes only with respect to the particular brief given to us, on the express condition that it will not be relied upon by any other person or for any other purposes without our prior written agreement, and relates to the conditions that exist up to and at the time of this report.

No liability is accepted by this firm or by any principal, or director, or any servant or agent of this firm, in respect of the use of this report by any other person, and any other person who relies upon any matter contained in this report does so entirely at its own risk. This disclaimer shall apply notwithstanding that this report may be made available to any person by any person in connection with any application for permission or approval, or pursuant to any requirement of law.

This report does not comment on stormwater management, flooding, root effects and land uses outside the specific site, which may be required to be assessed to complete a foundation design for building consent application purposes.

Notwithstanding the foregoing, if the circumstances at the subject site change with respect to topography or the proposed development concept, or the buildings are subject to further damaging earthquakes, or if a period of more than three years has elapsed since the date of this report, this report should not be used without our prior review and written agreement.

Notwithstanding the foregoing conclusions and recommendations, any proposed building development should be designed to satisfy the relevant requirements of the Building Code, so as to ensure compliance with the Building Act.

The conclusions and recommendations expressed herein should be read in conjunction with the remainder of this report and should not be referred to out of context with the remainder of this report.

Report prepared by: **FRASER THOMAS LTD.**	**Report reviewed and approved by:**
S P GLADWIN Engineering Geologist	**M V REED** Director Chartered Professional Engineer

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Appendix A

Field Investigation Results

Machine Excavated Test Pits

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BOREHOLE AND TEST PIT LOGS SYMBOLS AND TERMS

SYMBOLS AND ABBREVIATIONS

RL	Reduced Level
EOH	End of Hole
•	Shear vane test result
UTP	Unable to Penetrate
SPT	Standard Penetration Test
N	SPT blows per 300mm penetration
35/90	35 blows per 90mm penetration after seating for SPT
(s)	Inclusive of seating blow count for SPT
GWL	Ground Water Level

Wf	Field water content
Wp	Plastic limit (%)
WL	Liquid Limit (%)
RQD	Rock Quality Designation
SG	Specific Gravity
%F	Percentage fines (<75 microns)
PSD	Particle size distribution
CONS	Consolidation test
COMP	Compaction test
UCS	Unconfined Compressive Strength
k	Permeability coefficient (m/s)
LS	Linear Shrinkage (%)
OC	Organic Content (%)

SOIL

TOPSOIL
CLAY
SILT
SAND
GRAVEL
COBBLES
BOULDERS
PEAT
FILL

CONSISTENCY TERMS

Cohesive Description	Undrained Shear Strength (kPa)
Very Soft	<12
Soft	12 - 25
Firm	25 - 50
Stiff	50 - 100
Very Stiff	100 - 200
Hard	>200

RELATIVE DENSITY

Non-cohesive Description	SPT "N" Value
Very Loose	<4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	> 50

ROCK

LIMESTONE
MUDSTONE
SANDSTONE
CONGLOMERATE
BRECCIA
RYHOLITE
ANDESITE
BASALT

STRENGTH

Description	Unconfined Compressive Strength MPa
Extremely Weak	< 1
Very Weak	1 - 5
Weak	5 - 20
Moderately Strong	20 - 50
Strong	50 - 100
Very Strong	100 - 250
Extremely Strong	> 250

WEATHERING

UW - Unweathered (fresh rock)
SW - Slightly Weathered
MW - Moderately Weathered
HW - Highly Weathered
CW - Completely Weathered
RS - Residual Soil

SPACING OF DISCONTINUITIES

Term	Aperture (mm)
Very widely spaced	>2000
Widely spaced	600 to 2000
Moderately widely spaced	200 to 600
Closely spaced	60 to 200
Very closely spaced	20 to 60
Extremely closely spaced	<20

Notes

1. Based on New Zealand Geotechnical Society " Field Description of Soil and Rock,Guideline for the Field Classification and Description of Soil and Rock for Engineering Purposes" December 2005
2. Composite soil types are signified by combined symbols

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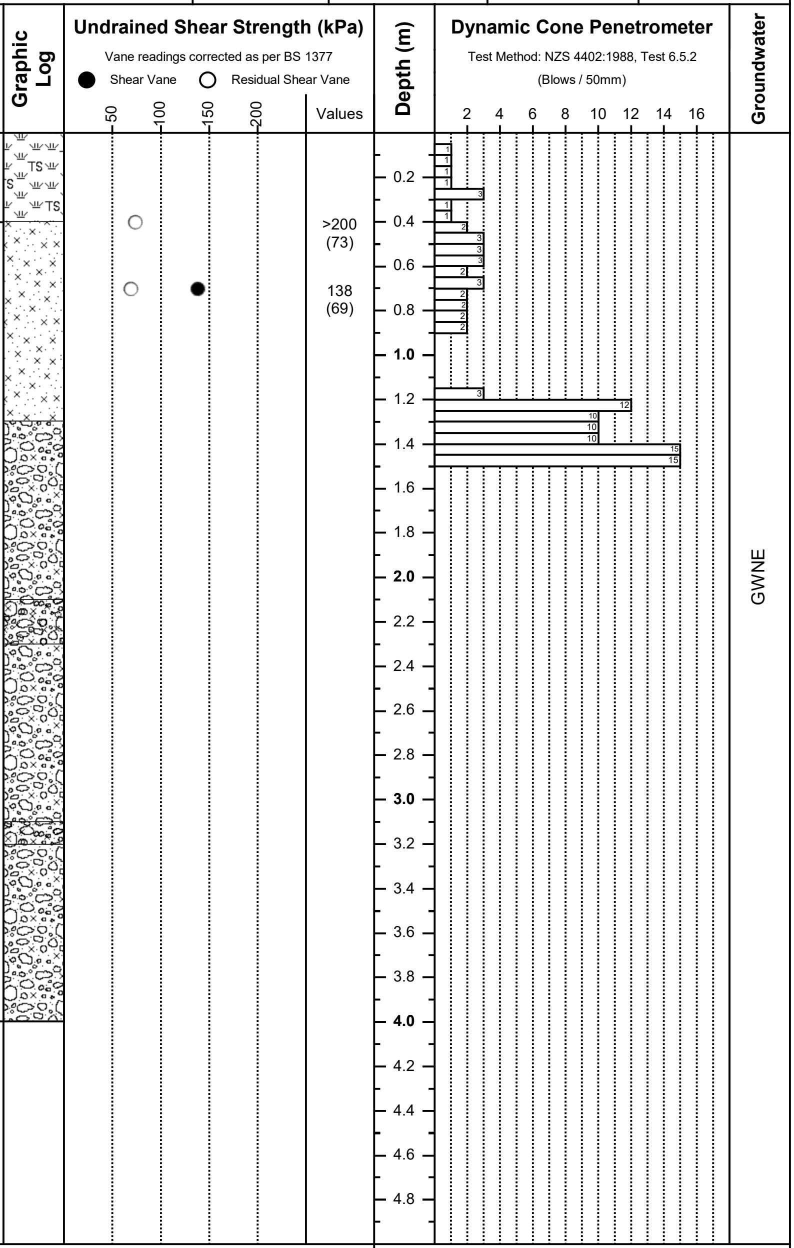
TEST PIT LOG

Hole No: TP1

Project No:	**Project:**	**Shear Vane:**	**Date Excavated:**	**Logged By:**	**Checked By:**
CH01763	Pajanti Ltd 259 Alford Forest Road, Ashburton	2512	03/08/2023	SG	

Description of Strata	Geological Unit
SILT, sandy, dark brown, moist, rootlets [TOPSOIL]	Topsoil
SILT, sandy, brown, very stiff, moist, low plasticity [ALLUVIAL SEDIMENTS]	Late Pleistocene Alluvial Sediments
GRAVEL (fine to coarse, subrounded), with some cobbles, sandy, silty, brown, very dense, moist 2.1 m - 2.3 m: sandy gravelly SILT, brown, moist 3.1 m - 3.2 m: sandy gravelly SILT, brown, moist	
EOTP: 4.00 m TARGET DEPTH	



Profile:

Excavation Method:

Remarks:

1. Groundwater not encountered on 03/08/2023.
2 Dynamic Cone Penetrometer (DCP) scala tests performed from the existing ground surface, and at a depth of approximately 1.1 m below the existing ground surface.

Datum:

Coordinates:

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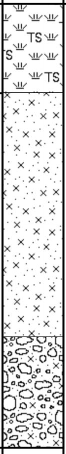
TEST PIT LOG

Hole No: **TP2**

Project No:	**Project:**	**Shear Vane:**	**Date Excavated:**	**Logged By:**	**Checked By:**
CH01763	**Pajanti Ltd** **259 Alford Forest Road, Ashburton**	2512	03/08/2023	SG	

Depth (m)	Description of Strata	Geological Unit
0.0 – 0.4	SILT, sandy, dark brown, moist, rootlets [TOPSOIL]	T/S
0.4 – 1.5	SILT, sandy, brown, very stiff, moist, low plasticity [ALLUVIAL SEDIMENTS]	Late Pleistocene Alluvial Sediments
1.5 – 2.0	GRAVEL (fine to coarse, subrounded), with some cobbles, sandy, silty, brown, very dense, moist	Late Pleistocene Alluvial Sediments
2.0	EOTP: 2.00 m TARGET DEPTH	

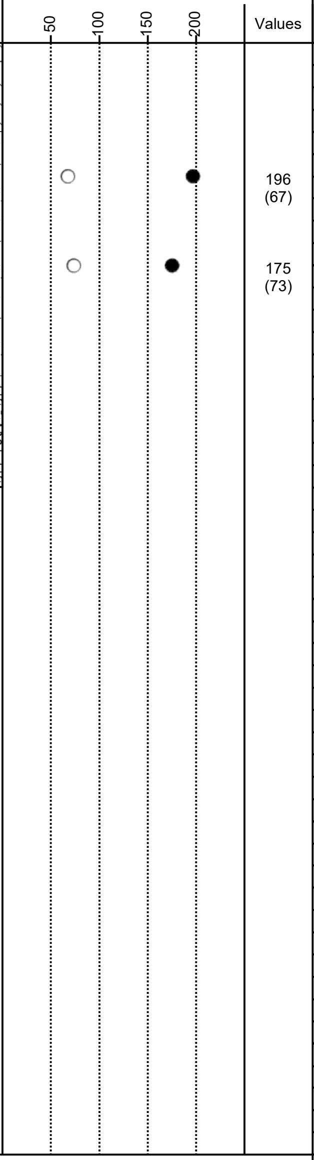
Graphic Log



Undrained Shear Strength (kPa)

Vane readings corrected as per BS 1377

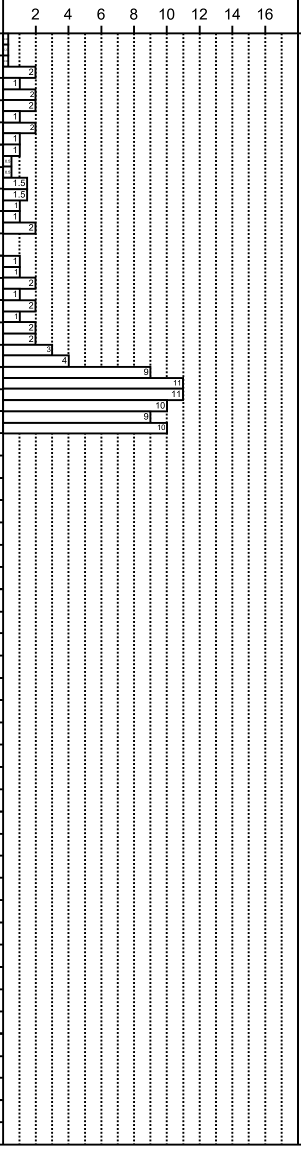
● Shear Vane ○ Residual Shear Vane



Dynamic Cone Penetrometer

Test Method: NZS 4402:1988, Test 6.5.2

(Blows / 50mm)



Groundwater: GWNE

Profile:

Excavation Method:

Remarks:

1. Groundwater not encountered on 03/08/2023.
2 Dynamic Cone Penetrometer (DCP) scala tests performed from the existing ground surface, and at a depth of approximately 0.9 m below the existing ground surface.

Datum:

Coordinates:

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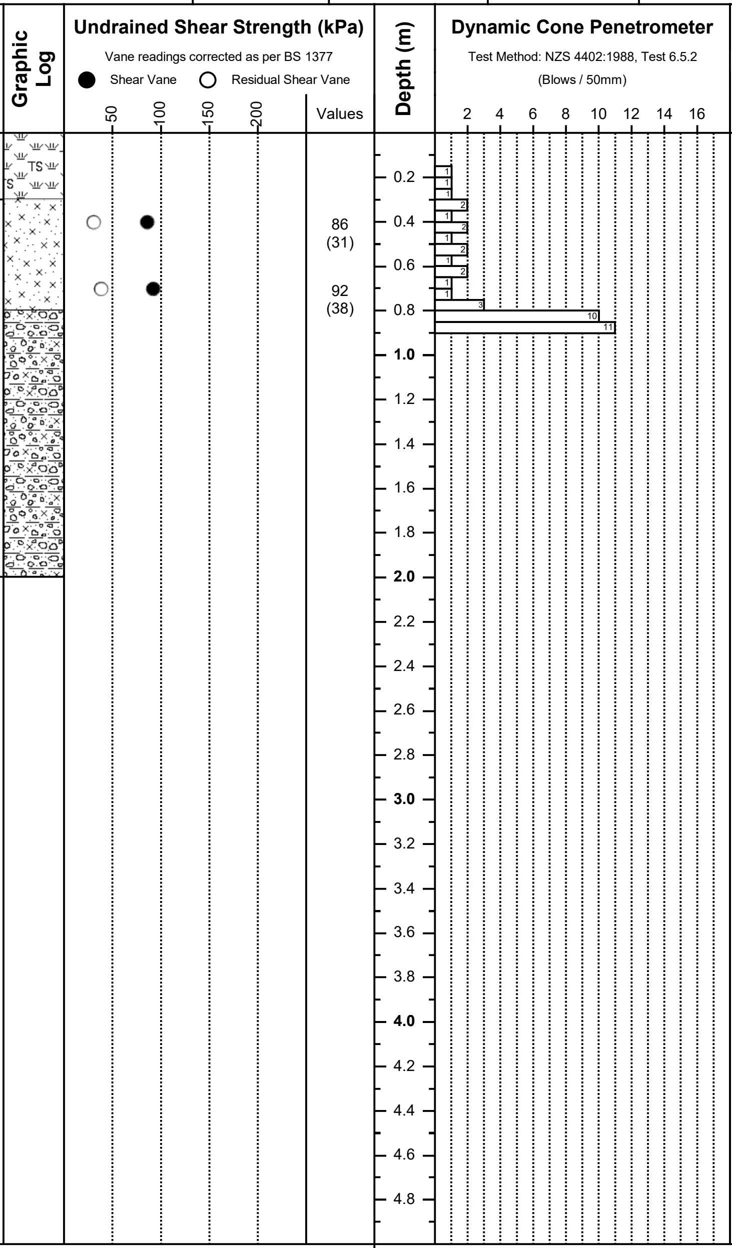
TEST PIT LOG

Hole No: TP3

Project No:	**Project:**	**Shear Vane:**	**Date Excavated:**	**Logged By:**	**Checked By:**
CH01763	Pajanti Ltd 259 Alford Forest Road, Ashburton	2512	03/08/2023	SG	

Depth (m)	Description of Strata	Geological Unit	Groundwater
0.0 – 0.3	SILT, sandy, dark brown, moist, rootlets [TOPSOIL]	Topsoil	GWNE
0.3 – 0.8	SILT, sandy, brown, very stiff, moist, low plasticity [ALLUVIAL SEDIMENTS]	Late Pleistocene Alluvial Sediments	
0.8 – 2.0	GRAVEL (fine to coarse, subrounded), with some cobbles, sandy, silty, brown, very dense, moist		
2.0	EOTP: 2.00 m TARGET DEPTH		



Profile:

Excavation Method:

Remarks:

1. Groundwater not encountered on 03/08/2023.

2 Dynamic Cone Penetrometer (DCP) scala test performed from the existing ground surface.

Datum:

Coordinates:

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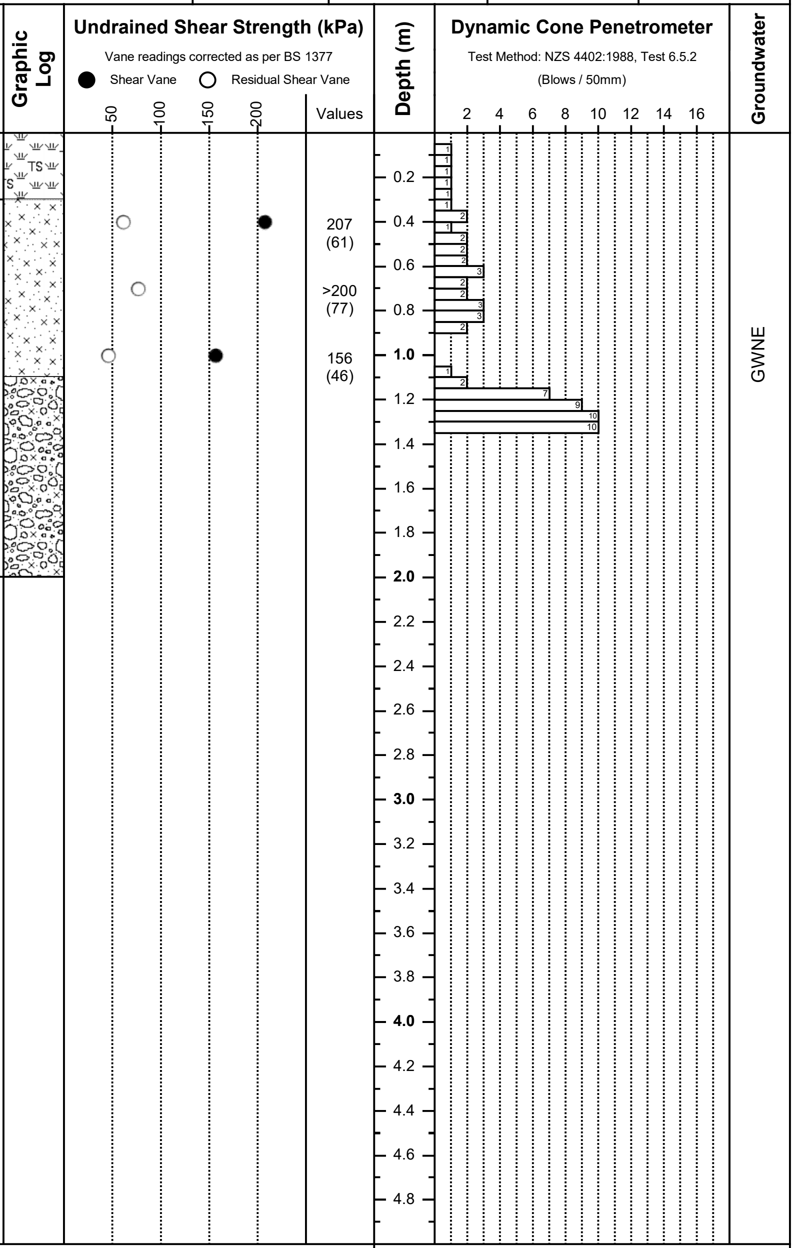
TEST PIT LOG

Hole No:
TP4

Project No:	**Project:**	**Shear Vane:**	**Date Excavated:**	**Logged By:**	**Checked By:**
CH01763	Pajanti Ltd 259 Alford Forest Road, Ashburton	2512	03/08/2023	SG	

Depth (m)	Description of Strata	Geological Unit
0.0 – 0.3	SILT, sandy, dark brown, moist, rootlets [TOPSOIL]	Topsoil
0.3 – 1.1	SILT, sandy, brown, very stiff, moist, low plasticity [ALLUVIAL SEDIMENTS]	Late Pleistocene Alluvial Sediments
1.1 – 2.0	GRAVEL (fine to coarse, subrounded), with some cobbles, sandy, silty, brown, very dense, moist	Late Pleistocene Alluvial Sediments
2.0	EOTP: 2.00 m TARGET DEPTH	



Profile:

Excavation Method:

Remarks:
1. Groundwater not encountered on 03/08/2023.
2 Dynamic Cone Penetrometer (DCP) scala tests performed from the existing ground surface, and at a depth of approximately 1.0 m below the existing ground surface.

Datum:

Coordinates:

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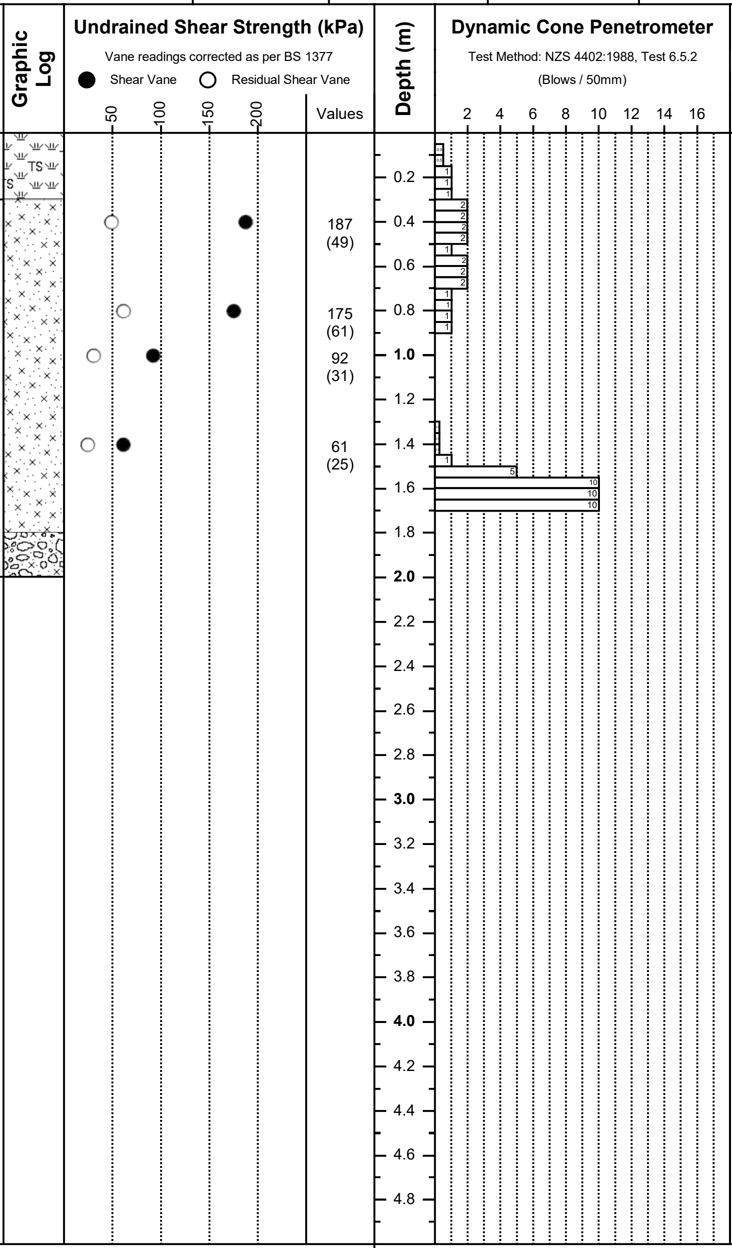
TEST PIT LOG

Hole No: TP5

Project No:	**Project:**	**Shear Vane:**	**Date Excavated:**	**Logged By:**	**Checked By:**
CH01763	Pajanti Ltd 259 Alford Forest Road, Ashburton	2512	03/08/2023	SG	

Depth (m)	Description of Strata	Geological Unit
0.0 – 0.3	SILT, sandy, dark brown, moist, rootlets [TOPSOIL]	T/S
0.3 – 1.8	SILT, sandy, brown, very stiff, moist, low plasticity [ALLUVIAL SEDIMENTS]	Late Pleistocene Alluvial Sediments
1.8 – 2.0	GRAVEL (fine to coarse, subrounded), with some cobbles, sandy, silty, brown, very dense, moist	Late Pleistocene Alluvial Sediments

EOTP: 2.00 m TARGET DEPTH



Depth (m)	Values
0.4	187 (49)
0.8	175 (61)
1.0	92 (31)
1.4	61 (25)

Groundwater: GWNE

Profile:

Excavation Method:

Remarks:
1. Groundwater not encountered on 03/08/2023.
2 Dynamic Cone Penetrometer (DCP) scala tests performed from the existing ground surface, and at a depth of approximately 1.3 m below the existing ground surface.

Datum:

Coordinates:

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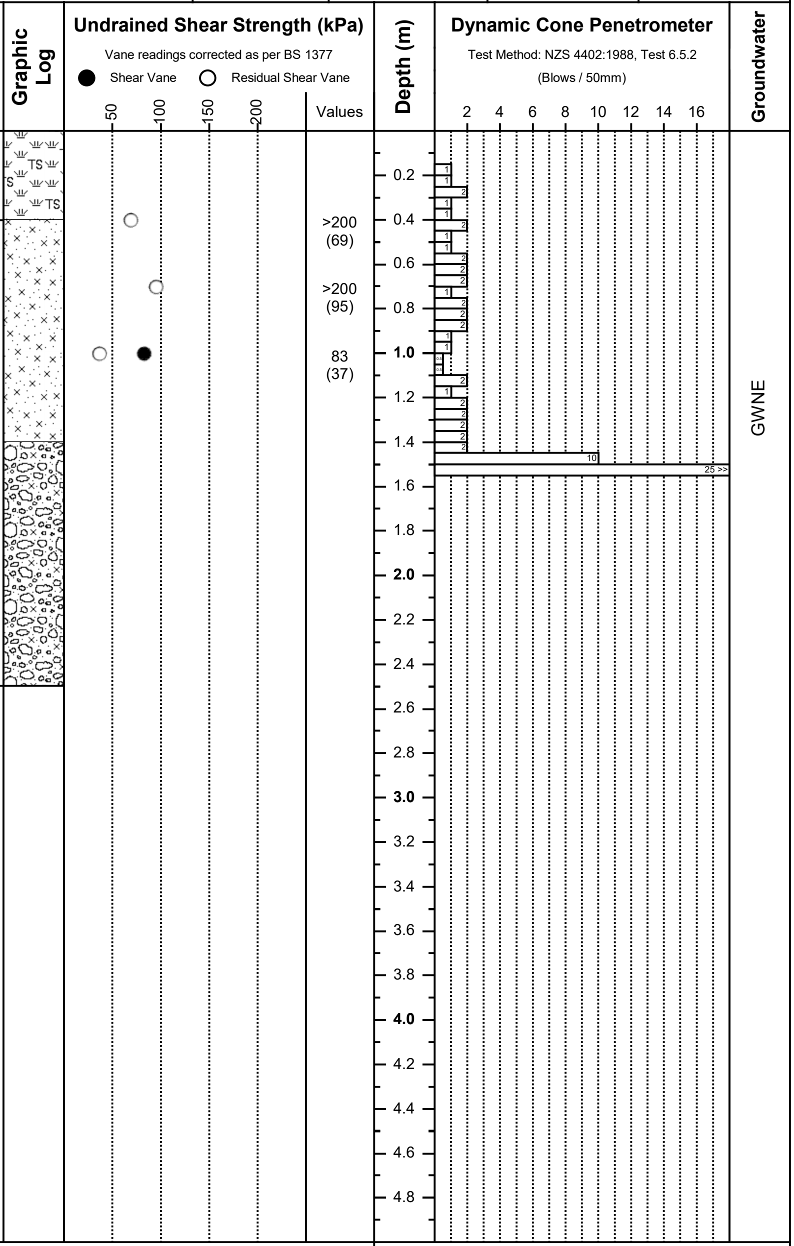
TEST PIT LOG

Hole No: **TP6**

Project No:	**Project:**	**Shear Vane:**	**Date Excavated:**	**Logged By:**	**Checked By:**
CH01763	**Pajanti Ltd** **259 Alford Forest Road, Ashburton**	2512	03/08/2023	SG	

Depth (m)	**Description of Strata**	**Geological Unit**
0.0 – 0.4	SILT, sandy, dark brown, moist, rootlets [TOPSOIL]	Topsoil
0.4 – 1.4	SILT, sandy, brown, very stiff, moist, low plasticity [ALLUVIAL SEDIMENTS]	Late Pleistocene Alluvial Sediments
1.4 – 2.5	GRAVEL (fine to coarse, subrounded), with some cobbles, sandy, silty, brown, very dense, moist	Late Pleistocene Alluvial Sediments
2.5	EOTP: 2.50 m TARGET DEPTH	



Profile:

Excavation Method:

Remarks:

1. Groundwater not encountered on 03/08/2023.
2 Dynamic Cone Penetrometer (DCP) scala tests performed from the existing ground surface, and at a depth of approximately 0.9 m below the existing ground surface.

Datum:

Coordinates:

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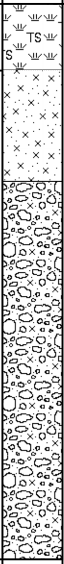
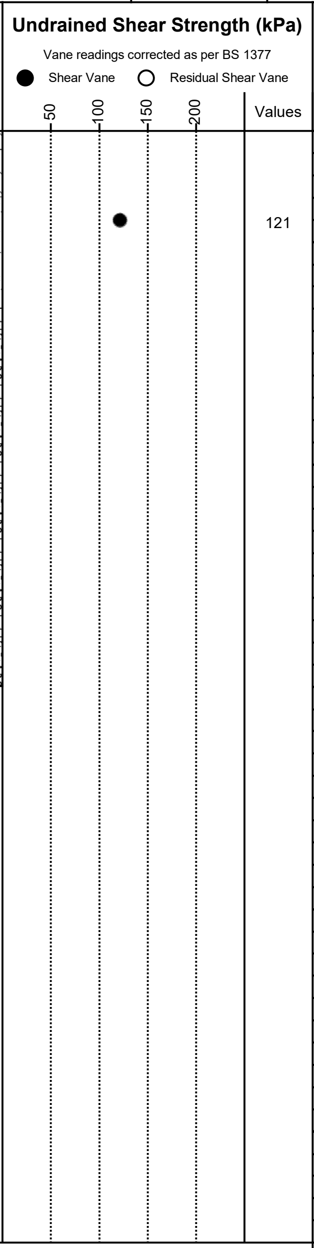
TEST PIT LOG

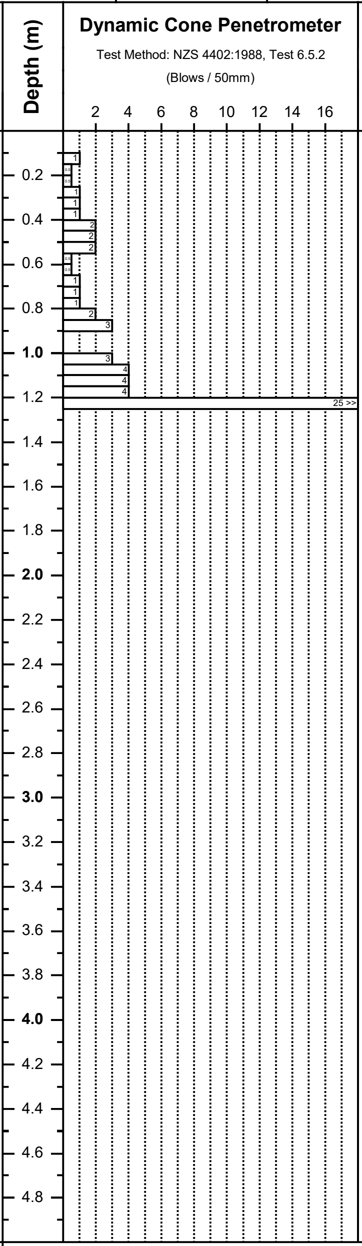
Hole No:
TP7

Project No:	**Project:**	**Shear Vane:**	**Date Excavated:**	**Logged By:**	**Checked By:**
CH01763	**Pajanti Ltd** **259 Alford Forest Road, Ashburton**	2512	03/08/2023	SG	

Description of Strata	**Geological Unit**	**Groundwater**
SILT, sandy, dark brown, moist, rootlets [TOPSOIL]	Topsoil	GWNE
SILT, sandy, brown, very stiff, moist, low plasticity [ALLUVIAL SEDIMENTS]	Late Pleistocene Alluvial Sediments	
GRAVEL (fine to coarse, subrounded), with some cobbles, sandy, silty, brown, very dense, moist		
EOTP: 2.50 m TARGET DEPTH		







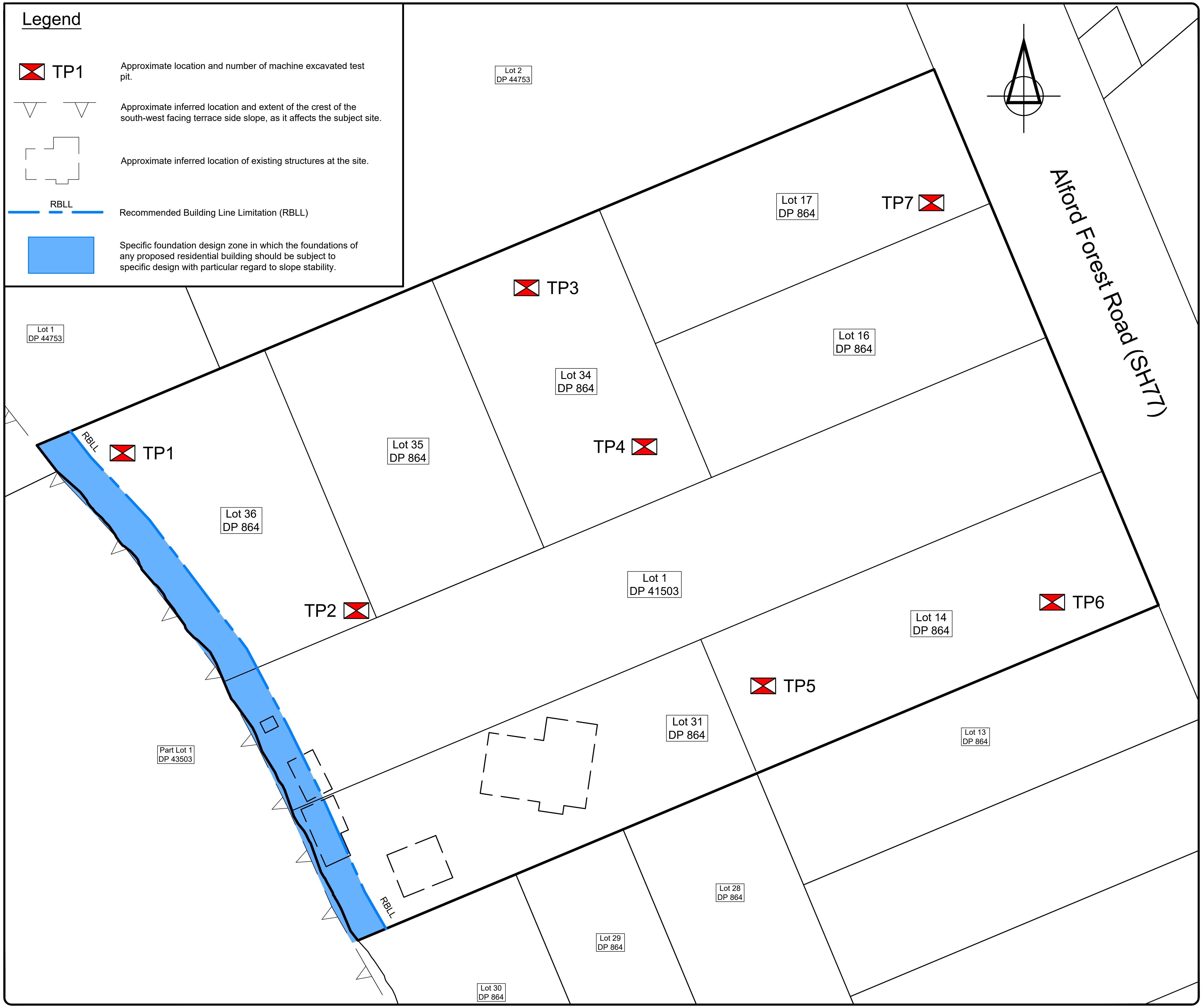
Profile:

Excavation Method:

Remarks:
1. Groundwater not encountered on 03/08/2023.
2 Dynamic Cone Penetrometer (DCP) scala tests performed from the existing ground surface, and at a depth of approximately 0.9 m below the existing ground surface.

Datum:

Coordinates:



Legend

☒ TP1	Approximate location and number of machine excavated test pit.
▽ ▽	Approximate inferred location and extent of the crest of the south-west facing terrace side slope, as it affects the subject site.
⌐ ¬	Approximate inferred location of existing structures at the site.
— — RBLL — —	Recommended Building Line Limitation (RBLL)
🟦	Specific foundation design zone in which the foundations of any proposed residential building should be subject to specific design with particular regard to slope stability.

			APPROVED	DATE
SURVEYED			MVR	
DESIGNED				
DRAWN				
CAD	GN	08/08/23		
CHECKED	SG	08/08/23		

REVISION	CHANGES	CHECKED	DATE

NOTES

1. This plan has been adopted from Grip map. The location and extent of the site boundaries and site features are therefore considered to be approximate only.
2. The location and extent of the south-west facing terrace side slope has been adopted from aerial photography and is therefore considered approximate only.

CLIENT

PAJANTI LTD

PROJECT

259 ALFORD FOREST ROAD,
ASHBURTON

TITLE

SITE PLAN

Fraser Thomas

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AUCKLAND	09 278 7078
HAWKE'S BAY	06 211 2766
CHRISTCHURCH	03 358 5936
BLENHEIM	03 428 3292
NELSON	03 222 1132
TAURANGA	020 4118 9465

www.fraserthomas.co.nz

SCALE 1:500 (A3)

DRAWING No REVISION

CH01763-G-01

SHEET 1 of 1