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PROPOSED SUBDIVISION AT
99 TAMUMU ROAD,
WAIPAWA

GEOTECHNICAL INVESTIGATION REPORT

DUNCAN
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PROPOSED SUBDIVISION AT 99 TAMUMU ROAD, WAIPAWA

GEOTECHNICAL INVESTIGATION REPORT

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SUMMARY

This report presents the results of a geotechnical investigation and appraisal undertaken for the proposed subdivision to be undertaken at 99 Tamumu Road, Waipawa.

It is understood that it is proposed to subdivide the site, in order to create five new residential lots (identified as proposed new Lots 1 to 5 inclusive). In order to mitigate the risk of flooding to the proposed development, it is our understanding that the client intends to 'build up' the site, which will involve the placement of engineered fill, to a depth of up to approximately 1.0 m above the existing ground surface.

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 Pleistocene age. These surficial soils are underlain by mudstone, inferred to be of the Whangai Formation.

Based on the results of the appraisal reported herein, it is our opinion that the subject site should be assumed with be within foundation Technical Category 1 (TC1), as defined by the MBIE Cantebury guidance document, and that it is unlikely that liquefaction induced ground deformation could occur at the site in response to a large earthquake event, and that the ground settlements at the site in response to seismic loading should be considered to be "within normally accepted tolerances" as defined by the MBIE Canterbury guidance document.

Given the nature and consistency of the soils underlying the subject site, it is our opinion that settlement of the founding soils, as a result of subdivisional fill loadings, is expected to be negligible, and should not present a problem for the proposed subdivision development at the site, providing the inspection and specific compaction of fill material is 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.

Foundation design parameters and recommendations are presented in Sections 9.0 and 10.0 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 the proposed residential development at the subject site.

The site is, in general, considered suitable for its intended use, with satisfactory conditions for the proposed subdivision 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 arising from the investigation are summarised in Section 14.0 of this report.

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99 TAMUMU ROAD,
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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation and appraisal undertaken for the proposed subdivision to be undertaken at 99 Tamumu Road, Waipawa.

The subject site is located on the southern side of Tamumu Road.

It is understood that it is proposed to subdivide the site, in order to create five new residential lots (identified as proposed new Lots 1 to 5 inclusive). In order to mitigate the risk of flooding to the proposed development, it is our understanding that the client intends to 'build up' the site, which will involve the placement of engineered fill, to a depth of up to approximately 1.0 m above the existing ground surface.

It is understood that a geotechnical report is required for subdivision application purposes, in order to assess the risk of the underlying subsoils being subject to consolidation settlement, due to loadings imposed by the proposed subdivisional filling.

The approximate inferred locations and extents of the proposed subdivision boundaries are shown on the appended Fraser Thomas Ltd drawing HB00059-G-01.

The subsurface conditions underlying the subject site have been investigated by means of four machine excavated test pits, numbered TP1 to TP4 inclusive.

A visual appraisal of the site, a review of the existing geotechnical information available for 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 the proposed subdivision, with particular regard to consolidation settlement and foundation design considerations, and to determine the suitability of the subject site for the proposed development, in support of an application for subdivision consent.

2.0 GEOLOGY

In assessing the geology of the site, reference has been made to the Geological and Nuclear Sciences, Map 8, Hawke's Bay Area, scale 1:250,000.

This map indicates that the site is likely to be underlain by alluvial sediments of Pleistocene age.

The geological map also indicates that the site is located near a geological boundary, with the Whangai Formation (typically consisting alternating mudstone, sandstone and shale, of late Cretaceous to Paleogene age), located in close proximity to the subject site. The Whangai Formation mudstone is expected to underlie the surficial soils at the site.

The results of the machine excavated test pit investigations reported herein, in general, indicate that the surficial soils underlying the site are likely to comprise alluvial sediments of Pleistocene age, which are, in turn, underlain by mudstone inferred to be of the Whangai Formation

3.0 PROPOSED DEVELOPMENT

It is understood that it is proposed to subdivide the site, in order to create five new residential lots (identified as proposed new Lots 1 to 5 inclusive). In order to mitigate the risk of flooding to the proposed development, it is our understanding that the client intends to 'build up' the site, which will involve the placement of engineered fill, to a depth of up to approximately 1.0 m above the existing ground surface.

The approximate inferred locations and extents of the proposed subdivision boundaries are shown on the appended Fraser Thomas Ltd drawing HB00059-G-01.

4.0 FIELD INVESTIGATION

4.1 GENERAL

The field investigation comprised a visual appraisal and four machine excavated test pits.

The approximate locations of the investigation test positions are shown on Fraser Thomas Ltd drawing HB00059-G-01.

4.2 RESULTS OF VISUAL APPRAISAL

A visual appraisal of the subject site was undertaken by a Fraser Thomas Ltd engineering geologist on 2 February 2024.

The subject site is located on the southern side of Tamumu Road.

The site topography is generally flat. A stormwater drain is located, and generally runs, along the southern boundary of the site. The northern bank of the drain is approximately 1.5 m in vertical height and slopes at approximately 45°.

The site was vacant and generally vegetated with paddock grass at the time of the investigation reported herein.

The approximate inferred locations and extents of the existing site features, are shown on the appended Fraser Thomas Ltd drawing HB00059-G-01.

4.3 MACHINE EXCAVATED TEST PIT INVESTIGATION

Four machine excavated test pits, numbered TP1 to TP4 inclusive, were excavated at the site, on the 2 February 2024. The machine excavated test pits were logged by a qualified Fraser Thomas engineering geologist.

The logs of the machine excavated test pits are presented in Appendix A of this report.

TP1 was undertaken to a target depth of approximately 3.6 m below the existing ground surface. TP2, TP3 and TP4 were terminated, due to encountering mudstone rock, at depths of between approximately 2.9 m and 3.5 m below the existing ground surface.

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

Dynamic Cone Penetration (DCP) tests were undertaken within the non-cohesive sediments.

The approximate locations of the machine excavated test pits are shown on drawing HB00059-G-01.

5.0 SUBSURFACE CONDITIONS

5.1 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 Pleistocene age. These surficial soils are underlain by mudstone, inferred to be of the Whangai Formation.

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.

5.2 TOPSOIL

A surficial layer of topsoil, generally comprising clayey silts, was encountered to depths of between approximately 0.2 m and 0.3 m below the existing ground surface, at the locations of the machine excavated test pits.

5.3 ALLUVIAL SEDIMENTS

5.3.1 Silty Clays

The results of the machine excavated test pit investigation indicates that the surficial topsoil is, in general, underlain by material generally comprising non-organic silty clays, inferred to be alluvial sediments of Pleistocene age.

A minor lens (generally less than 100 mm thickness) of slightly organic silt material was encountered at the locations of TP1, TP2 and TP3, at a depth of approximately 1.0 m below the existing ground surface.

The surficial cohesive soils were generally encountered to depths of between approximately 1.9 m and 2.4 m below the existing ground surface, at the locations of the machine excavated test pits.

In situ undrained shear strength values of between approximately 103 kPa and greater than 200 kPa, were generally measured within the cohesive soils, using hand held shear vane equipment, corresponding to a very stiff to hard consistency.

5.3.2 Sandy Gravels

A layer of material, generally comprising sandy gravels, inferred to be alluvial sediments was encountered below the surficial layer of cohesive soils, at depths ranging between approximately 1.9 m and 2.4 m below the existing ground surface. These soils were encountered to depths of between approximately 2.8 m and 3.4 m below the existing ground surface.

The results of the DCP tests, undertaken in the upper layers of sandy gravels, generally obtained DCP blow counts of between approximately 6 and greater than 20 blows per 100 mm of penetration, typically corresponding to a very dense consistency.

5.4 WHANGAI FORMATION

The results of the machine excavated test pit investigation indicate that surficial alluvial sediments are underlain by slightly weathered mudstone, inferred to be of the Whangai Formation. This bedrock was encountered at depths ranging between approximately 2.8 m and 3.4 m below the existing ground surface.

The rock has been assessed as being ‘very weak’ in accordance with field identification criterion described in the New Zealand Geotechnical Society “Field Description of Soil and Rock”.

5.5 GROUNDWATER

Based on the results of the machine excavated test pit investigations reported herein, the groundwater level is inferred to be at a depth of approximately 2.5 m below the existing ground surface, for analysis purposes.

6.0 SITE SUBSOIL CLASS

The ‘Site Subsoil Class’ at the site has been evaluated based on the classification criteria provided in Section 3.1.3 of NZS 1170.5:2004, Structural design actions (Part 5).

Based on the investigations undertaken by Fraser Thomas and presented herein, we consider that the site can be classified as ‘Class C – Shallow Soil’, as defined by NZS 1170.5.

7.0 LIQUEFACTION POTENTIAL ASSESSMENT

7.1 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.

7.2 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.

7.3 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 2.0 of this report, the geological map for the Hawke's Bay area indicates that the site is likely to be underlain by alluvial sediments of Pleistocene age.

As discussed in Sections 5.3 and 5.4 of this report, the results of the machine excavated test pit investigations reported herein indicate the surficial layer of topsoil at the site is generally underlain by material comprising silty clays and sandy gravels, which are underlain by weathered mudstone, inferred to be of the Whangai Formation, of late Cretaceous to Paleogene age.

As discussed in Section 5.5 of this report, the groundwater level is inferred to be at a depth of approximately 2.5 m below the existing ground surface, for analysis purposes.

Based on the foregoing, given the age, nature and consistency of the sediments underlying the subject site. i.e. typically unsaturated cohesive soils, underlain by very dense gravels of Pleistocene age (in turn underlain by mudstone), it is our opinion that the upper 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 Canterbury 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.

8.0 SETTLEMENT CONSIDERATIONS

As discussed in Section 1.0 of this report, in order to mitigate the risk of flooding to the proposed development, it is our understanding that the client intends to 'build up' the site, which will involve the placement of engineered fill, to a depth of up to approximately 1.0 m above the existing ground surface.

As discussed in Section 5.0 of this report, the results of our site-specific geotechnical investigations indicate that the subject site is underlain by a surficial layer of very stiff to hard silty clays, which is, in turn, underlain by a layer of very dense sandy gravels, which is, in turn, underlain by weathered mudstone bedrock.

Given the nature and consistency of the soils underlying the subject site, it is our opinion that settlement of the founding soils, as a result of subdivisional fill loadings, is expected to be negligible, and should not present a problem for the proposed subdivision development at the site, providing the inspection and specific compaction of fill material is 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.

Unless further specific appraisal is undertaken, any fill placed above the existing ground surface, to mitigate against the risk of flooding of the proposed development, should be no greater than approximately 1.0 m in thickness.

9.0 FOUNDATION DESIGN CONSIDERATIONS

9.1 GENERAL

It is our opinion that the alluvial 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, and the proposed subdivisional fill loadings.

It is, therefore, our opinion that settlement should not present a problem for the proposed subdivision 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.

9.2 LIQUEFACTION-INDUCED GROUND DEFORMATIONS

As discussed in Section 7.3 of this report, it is our opinion that the subject site should be assumed with be within foundation Technical Category 1 (TC1), as defined by the MBIE Cantebury guidance document, and that it is unlikely that liquefaction induced ground deformation could occur at the site in response to a large earthquake event, and that the ground settlements at the site in response to seismic loading should be considered to be “within normally accepted tolerances” as defined by the MBIE Canterbury guidance document.

Based on the results of the investigation and appraisal reported herein, it is our opinion that an appropriate foundation design for the site conditions would be a shallow foundation system designed in accordance with the requirements of NZS 3604:2011 (as modified by B1/AS1), and in accordance with the recommendations presented in this report.

9.3 FOUNDATIONS IN CLOSE PROXIMITY TO PROPOSED FILL END SLOPES

As discussed in Section 1.0 of this report, in order to mitigate the risk of flooding to the proposed development, it is our understanding that the client intends to ‘build up’ the site, which will involve the placement of engineered fill, to a depth of up to approximately 1.0 m above the existing ground surface.

There is a risk that shallow foundations, located close to the crests of fill end slopes, may be adversely affected by soil creep movement. Based on the foregoing, and unless Fraser Thomas Ltd are engaged to undertake further appraisal works, it is recommended that the outside edge of any proposed shallow perimeter beam foundation, be located no closer than a horizontal distance of approximately 1.5 m from the crest of any fill end slope (no greater than 1.0 m in vertical height).

10.0 ALLOWABLE FOUNDATION BEARING PRESSURES

10.1 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.

10.2 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 or engineered fill (at a minimum depth of approximately 0.3 m below the finished ground surface). 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 or engineered fill (at a minimum depth of approximately 0.3 m below the finished ground surface).

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

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

11.0 EARTHWORKS CONSIDERATIONS

It is recommended that any proposed subdivisional bulk fill, placed to ‘build up’ the site, be “engineered fill”, which involves the specific placement and compaction of clean fill material. The proposed fill will need to be subject to appropriate observation and testing, by a suitably qualified CPEng engineer, so as to confirm that the fill has been appropriately placed and compacted and that the fill is suitable for its intended purpose.

It is recommended that any topsoil material be stripped from the surface of any subgrade, prior to the placement of any building platform fill material.

Fraser Thomas Ltd should be engaged to inspect the stripped subgrade, prior to the placement of any fill materials, in order to confirm that the subgrade is suitable for the placement of any fill.

The compaction standards for engineered fill material for residential development should meet the following requirements as determined by the methods of NZS 4402: 1986; New Zealand Standard Methods of Testing Soils for Civil Engineering Purposes, and BS 1377: 1975; British Standard Methods of Test for Soils for Civil Engineering Purposes:

(a) An average air voids value of not more than 10 % and any one test site value of not more than 12 %.

(b) An average in situ undrained shear strength value of not less than 120 kPa and any one test site value of not less than 100 kPa.

It is recommended that Fraser Thomas be engaged to inspect and test any fill material, so that the adequacy of the fill can be verified.

Unless further specific appraisal is undertaken, any fill placed above the existing ground surface, to mitigate against the risk of flooding of the proposed development should be no greater than 1.0 m in thickness.

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° (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 CONCLUSIONS AND RECOMMENDATIONS

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

14.1 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 the proposed residential development at the subject site.

The site is, in general, considered suitable for its intended use, with satisfactory conditions for the proposed subdivision 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.

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 proposed development.

(b) 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 Pleistocene age. These surficial soils are underlain by mudstone, inferred to be of the Whangai Formation.

(c) The 'Site Subsoil Class' at the site has been evaluated based on the classification criteria provided in Section 3.1.3 of NZS 1170.5:2004, Structural design actions (Part 5). Based on the investigations undertaken by Fraser Thomas reported herein, we consider that the site can be classified as 'Class C – Shallow Soil', as defined by NZS 1170.5.

(d) Based on the foregoing, given the age, nature and consistency of the sediments underlying the subject site. i.e. typically unsaturated cohesive soils, underlain by very dense gravels of Pleistocene age (in turn underlain by mudstone), it is our opinion that the upper 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 Canterbury 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.

(e) As discussed in Section 5.0 of this report, the results of our site-specific geotechnical investigations indicate that the subject site is underlain by a surficial layer of very stiff to hard silty clays, which is, in turn, underlain by a layer of very dense sandy gravels, which is, in turn, underlain by weathered mudstone bedrock.

Given the nature and consistency of the soils underlying the subject site, it is our opinion that settlement of the founding soils, as a result of subdivisional fill loadings, is expected to be negligible, and should not present a problem for the proposed subdivision development at the site, providing the inspection and specific compaction of fill material is 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.

Unless further specific appraisal is undertaken, any fill placed above the existing ground surface, to mitigate against the risk of flooding of the proposed development, should be no greater than 1.0 m in thickness.

(f) It is our opinion that the alluvial 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, and the proposed subdivisional fill loadings.

It is, therefore, our opinion that settlement should not present a problem for the proposed subdivision 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.

(g) It is our opinion that an appropriate foundation design for the site conditions would be a shallow foundation system designed in accordance with the requirements of NZS 3604:2011 (as modified by B1/AS1), and in accordance with the recommendations presented in this report.

14.2 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) There is a risk that shallow foundations, located close to the crests of fill end slopes, may be adversely affected by soil creep movement. Based on the foregoing, and unless Fraser Thomas Ltd are engaged to undertake further appraisal works, it is recommended that the outside edge of any proposed shallow perimeter beam foundation, be located no closer than a horizontal distance of approximately 1.5 m from the crest of any fill end slope (no greater than 1.0 m in vertical height).

(b) 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 or engineered fill (at a minimum depth of approximately 0.3 m below the finished ground surface). 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.

(c) It is recommended that any proposed subdivisional bulk fill, placed to ‘build up’ the site, be “engineered fill”, which involves the specific placement and compaction of clean fill material. The proposed fill will need to be subject to appropriate observation and testing, by a suitably qualified CPEng engineer, so as to confirm that the fill has been appropriately placed and compacted and that the fill is suitable for its intended purpose.

(d) It is recommended that any topsoil material be stripped from the surface of any subgrade, prior to the placement of any building platform fill material. Fraser Thomas Ltd should be engaged to inspect the stripped subgrade, prior to the placement of any fill materials, in order to confirm that the subgrade is suitable for the placement of any fill.

(e) The compaction standards for engineered fill material for residential development should meet the following requirements as determined by the methods of NZS 4402: 1986; New Zealand Standard Methods of Testing Soils for Civil Engineering Purposes, and BS 1377: 1975; British Standard Methods of Test for Soils for Civil Engineering Purposes:

  (i) An average air voids value of not more than 10 % and any one test site value of not more than 12 %.

  (ii) An average in situ undrained shear strength value of not less than 120 kPa and any one test site value of not less than 100 kPa.

(f) It is recommended that Fraser Thomas be engaged to inspect and test any fill material, so that the adequacy of the fill can be verified.

(g) 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.

(h) 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.

15.0 LIMITATIONS

The professional opinion expressed herein has been prepared solely for, and is furnished to our client, Duncan Leighton, and Central Hawke's Bay 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.

Notwithstanding the foregoing, if the circumstances at the subject site change with respect to topography or the proposed development concept, 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.

L H COWAN
Engineering Geologist

Report reviewed and approved by:

M V REED
Director
Chartered Professional Engineer

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Appendix A

Field Investigation Results

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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
TDTA	Too Difficult to Auger
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 - 2000
Moderately widely spaced	200 - 600
Closely spaced	60 - 200
Very closely spaced	20 - 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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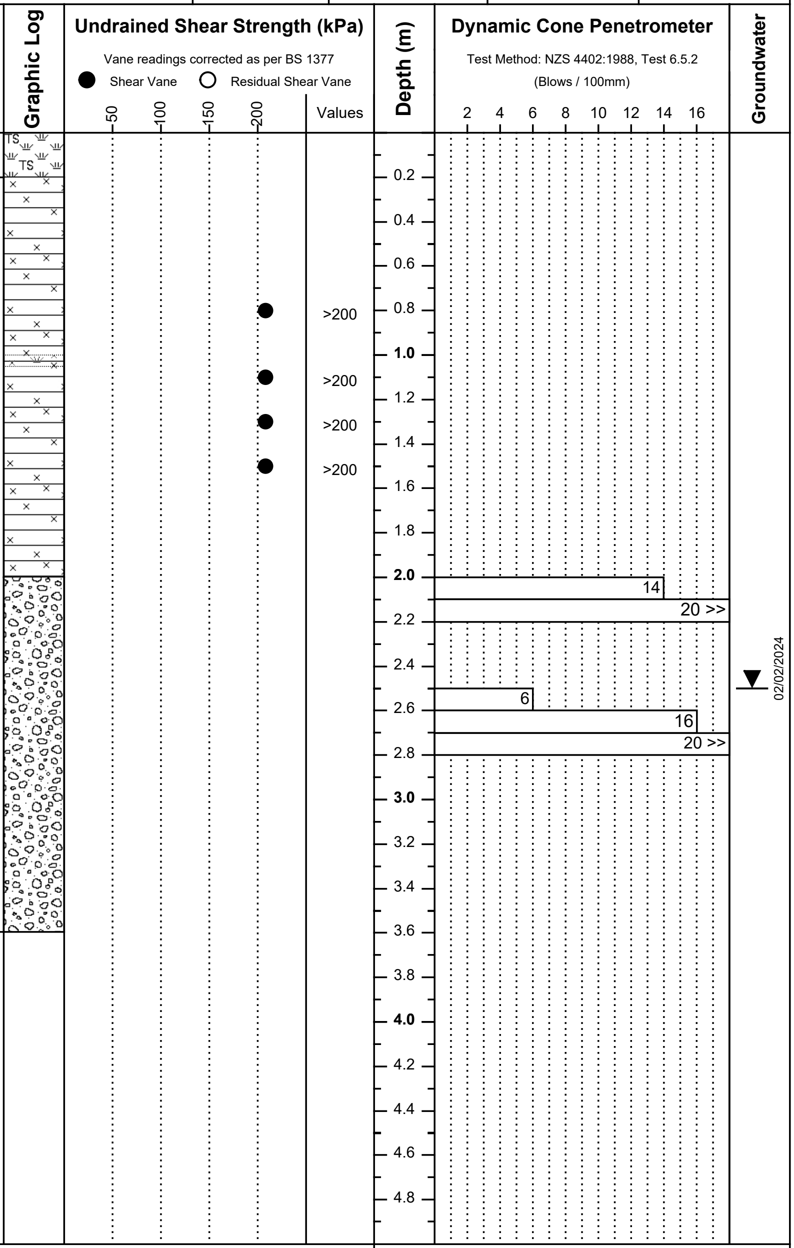
INVESTIGATION LOG

Hole No: TP1

Project No:	Project:	Shear Vane:	Date Excavated:	Logged By:	Checked By:
HB00059	Duncan Leighton 99 Tamumu Road, Waipawa	2699	02/02/2024	LHC	MVR

Depth (m)	Description of Strata	Geological Unit
0.0 – 0.2	SILT, clayey, dark brown, stiff, moist, slight plasticity [TOPSOIL]	T/S
0.2 – 2.0	CLAY, silty, light grey with orange streaks, hard, moist, moderate to high plasticity [PLEISTOCENE ALLUVIAL DEPOSITS] 1.0 m - 1.1 m: slightly organic silt	Pleistocene Alluvial Deposits
2.0 – 3.6	GRAVEL (fine to medium, rounded to sub-rounded), sandy, bluish grey, medium dense to very dense, moist, non-plastic 2.5 m: becoming saturated	
3.6	EOTP: 3.60 m Target depth	



Profile:

Excavation Method:

Remarks:
Standing groundwater encountered at approximately 2.5 m on the 2/02/2024

Datum:

Coordinates:

Generated with CORE-GS by Geroc - Shallow Investigation with Photo EOTP test - 11/03/2024 4:37:07 pm

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INVESTIGATION LOG

Hole No: TP2

Project No:	**Project:**	**Shear Vane:**	**Date Excavated:**	**Logged By:**	**Checked By:**
HB00059	Duncan Leighton 99 Tamumu Road, Waipawa	2699	02/02/2024	LHC	MVR

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 / 100mm)

Depth (m)	Description of Strata	Geological Unit	Undrained Shear Strength Values (kPa)
0.0 – 0.3	SILT, clayey, dark brown, stiff, moist, slight plasticity [TOPSOIL]	T/S	
0.3 – 1.0	CLAY, light blueish grey, very stiff, moist, high plasticity [PLEISTOCENE ALLUVIAL DEPOSITS]	Pleistocene Alluvial Deposits	166 (66)
1.0 – 1.3	SILT, brown, stiff, moist, moderate plasticiy, slightly organic	Pleistocene Alluvial Deposits	>200
1.3 – 2.1	CLAY, silty, light grey with orange streaks, hard, moist, moderate to high plasticity	Pleistocene Alluvial Deposits	>200
2.1 – 3.4	GRAVEL (fine to medium, rounded to sub-rounded), sandy, bluish grey, medium dense to very dense, moist, non-plastic 2.7 m: becoming saturated	Pleistocene Alluvial Deposits	
3.4 – 3.5	MUDSTONE, slightly weathered, grey, very weak, closely spaced discontinuities [WHANGAI FORMATION]		

EOTP: 3.50 m Target depth

Depth (m)	DCP (Blows / 100mm)
2.1 – 2.2	7
2.2 – 2.3	17
2.3 – 2.4	20 >>
2.8 – 2.9	13
2.9 – 3.0	8
3.0 – 3.1	9
3.1 – 3.2	20 >>

Groundwater: 02/02/2024 (approx. 2.7 m)

Profile:

Excavation Method:

Remarks:
Standing groundwater encountered at approximately 2.7 m on the 2/02/2024

Datum:

Coordinates:

Generated with CORE-GS by Geroc - Shallow Investigation with Photo EOTP test - 11/03/2024 4:37:14 pm

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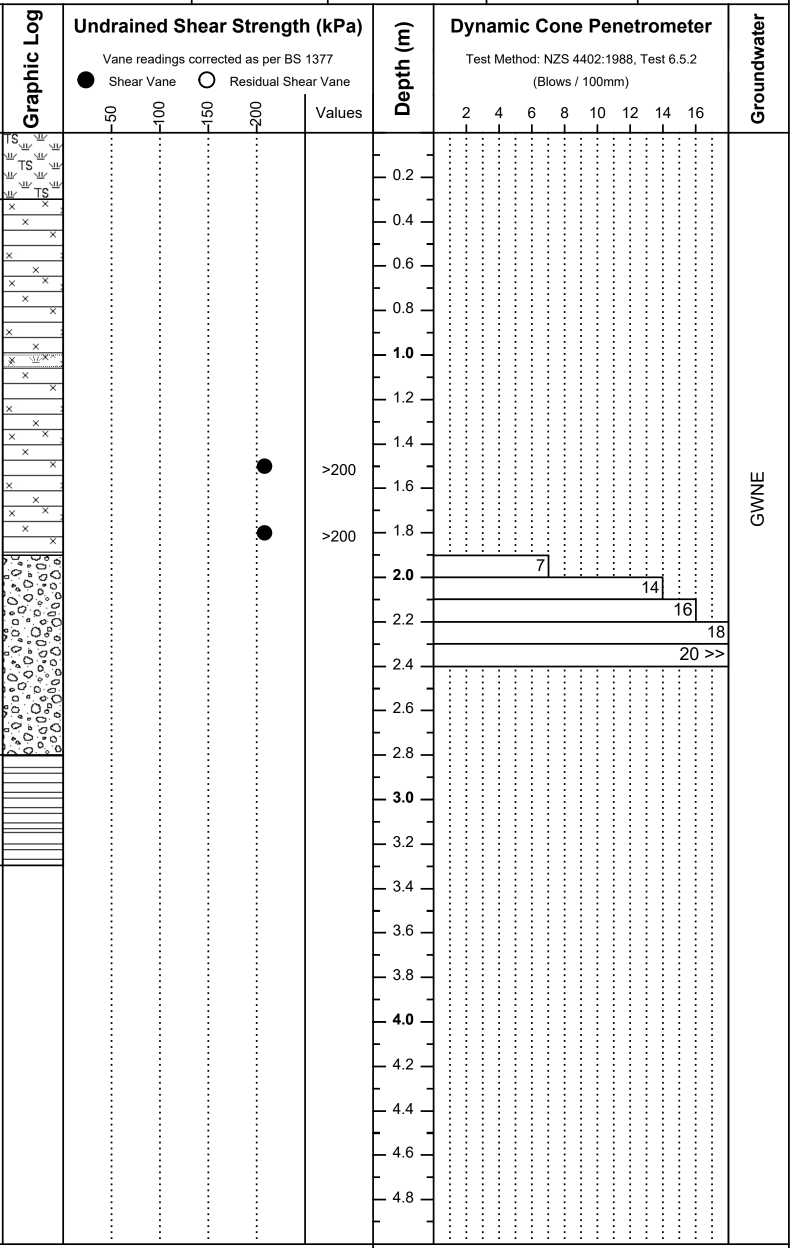
INVESTIGATION LOG

Hole No: TP3

Project No:	Project:	Shear Vane:	Date Excavated:	Logged By:	Checked By:
HB00059	Duncan Leighton 99 Tamumu Road, Waipawa	2699	02/02/2024	LHC	MVR

Description of Strata	Geological Unit
SILT, clayey, dark brown, stiff, moist, slight plasticity [TOPSOIL]	T/S
CLAY, silty, light grey with orange streaks, hard, moist, moderate to high plasticity [PLEISTOCENE ALLUVIAL DEPOSITS] 1.0 m - 1.1 m: slightly organic silt	Pleistocene Alluvial Deposits
GRAVEL (fine to medium, rounded to sub-rounded), sandy, bluish grey, medium dense to very dense, moist, non-plastic	Pleistocene Alluvial Deposits
MUDSTONE, slightly weathered, grey, very weak, closely spaced discontinuities [WHANGAI FORMATION]	Whangai Formation
EOTP: 3.30 m Unable to proceed due to encountering rock	



Profile:

Excavation Method:

Remarks:
Standing groundwater not encountered during investigation on the 2/02/2024

Datum:

Coordinates:

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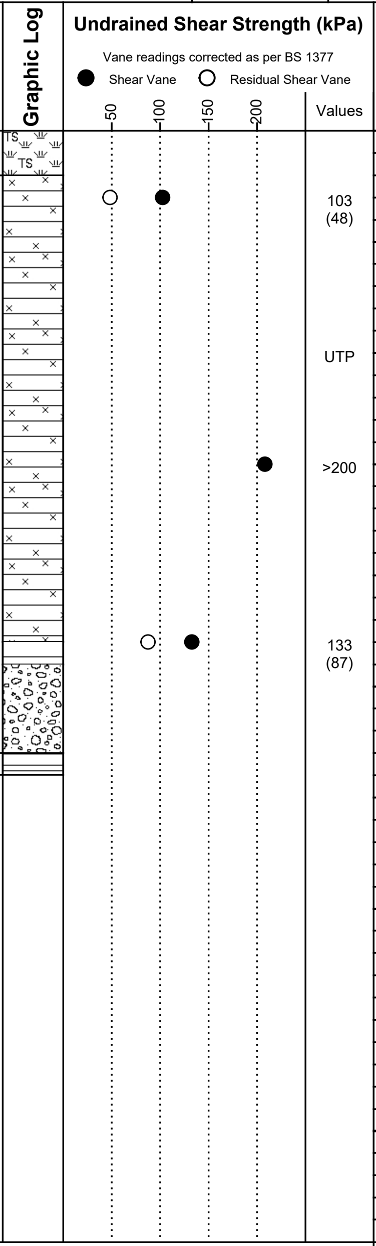
INVESTIGATION LOG

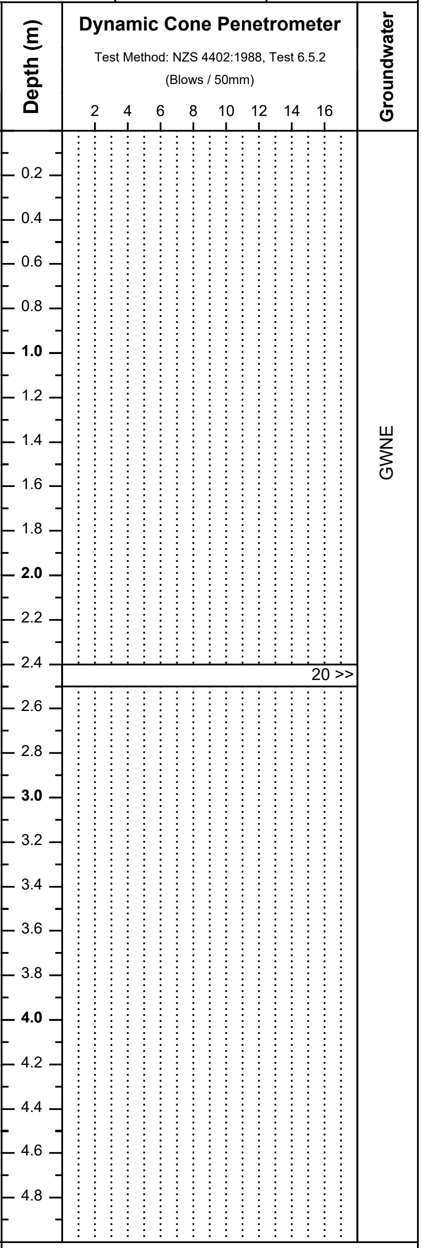
Hole No: TP4

Project No:	**Project:**	**Shear Vane:**	**Date Excavated:**	**Logged By:**	**Checked By:**
HB00059	Duncan Leighton 99 Tamumu Road, Waipawa	2699	02/02/2024	LHC	MVR

Depth (m)	Description of Strata	Geological Unit
0.2	SILT, clayey, dark brown, stiff, moist, slight plasticity [TOPSOIL]	T/S
0.4 – 2.2	CLAY, silty, light grey with orange streaks, very stiff to hard, moist, moderate to high plasticity [PLEISTOCENE ALLUVIAL DEPOSITS]	Pleistocene Alluvial Deposits
2.4	CLAY with trace organics, bluish grey, very stiff, moist, high plasticity	
2.6 – 2.8	GRAVEL (fine to medium, rounded to sub-rounded), sandy, bluish grey, medium dense to very dense, moist, non-plastic	
3.0 – 3.2	MUDSTONE, slightly weathered, grey, very weak, closely spaced discontinuities [WHANGAI FORMATION]	

EOTP: 2.90 m Unable to proceed due to encountering rock





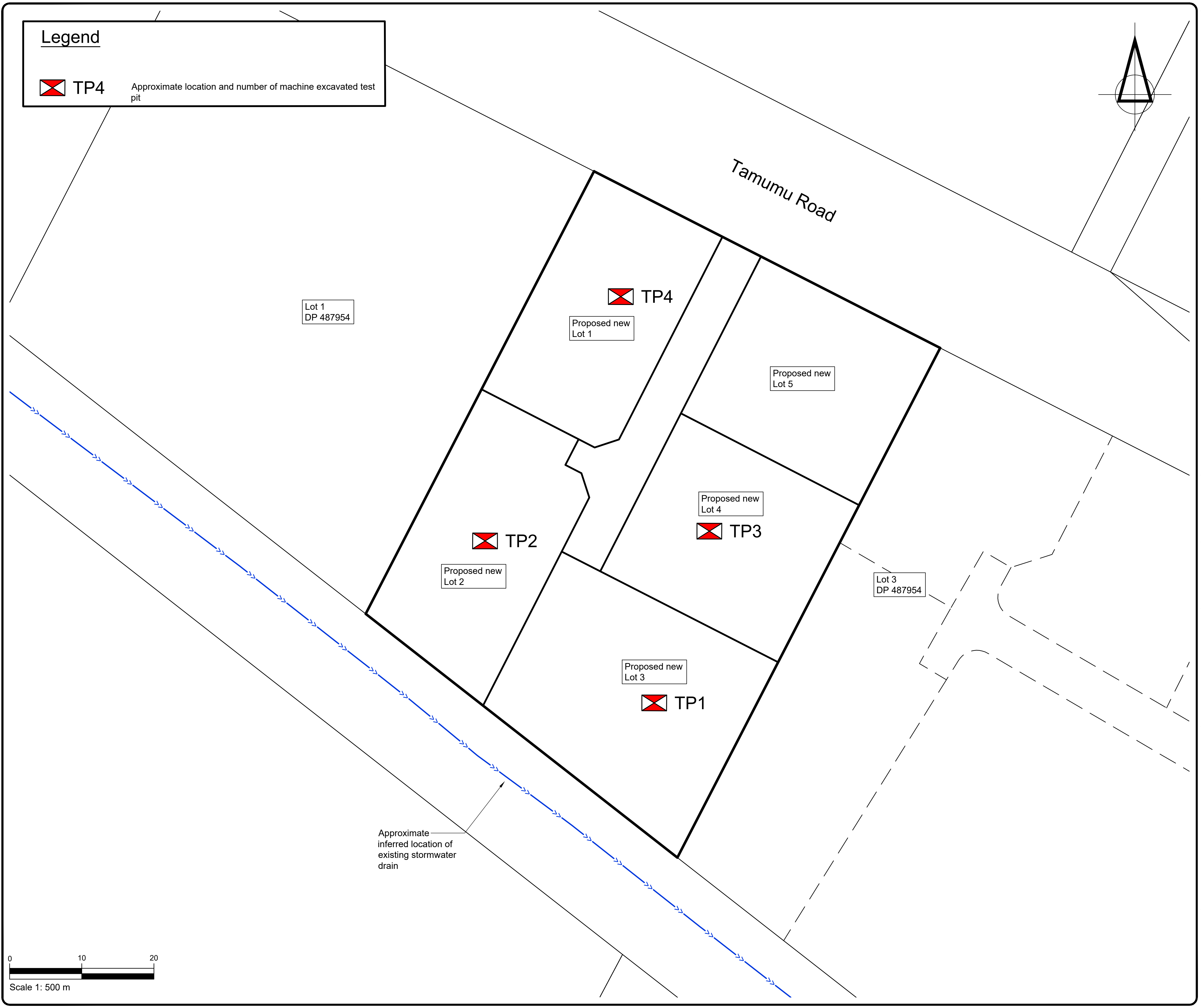
Profile:

Excavation Method:

Remarks:
Standing groundwater not encountered during investigation on the 2/02/2024

Datum:

Coordinates:



X:\HB00059 LEIGHTON 99 Tamumu Rd Waipawa\Geotechnical\01 CAD\03 Drawings\HB00059-G-01.dwg, lcowan,11/10/2021 10:18 am

SURVEYED		
DESIGNED		
DRAWN	GN	22/02/24
CHECKED	LC	22/02/24

APPROVED DATE

MVR

REVISION	CHANGES	CHECKED	DATE

NOTES

1. This plan has been adopted from Grip Map. The locaton and extent of site boundaries and site features are therefore considered approximate only.
2. The location of the proposed subdivision boundaries have been adopted based on the Scheme plan recieved from Development Nous dated 10/03/2021

CLIENT

DUNCAN LEIGHTON

PROJECT

99 TAMUMU ROAD,
WAIPAWA

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

www.fraserthomas.co.nz

STATUS

Construction works shall commence only on receipt of and in accordance with the Council or Council organisation stamped approved drawings, unless otherwise indicated.

SCALE 1:500 (A3)

DRAWING No HB00059-G-01

REVISION -