# Annexure 9:

# Geotechnical Assessment

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ENGINEERS • RESOURCE MANAGERS • SURVEYORS

PROPOSED PLAN CHANGE-
DARFIELD PREFERRED
DEVELOPMENT AREA 7
(MALVERN AREA PLAN)

# GEOTECHNICAL INVESTIGATION REPORT

Mr Mervyn Todd

# PROPOSED PLAN CHANGE- DARFIELD PREFERRED DEVELOPMENT AREA 7 (MALVERN AREA PLAN)

# GEOTECHNICAL INVESTIGATION REPORT

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## SUMMARY

This report presents the results of a geotechnical investigation and appraisal undertaken for the site at Lot 4 DP 524058, Pt RS 27204 and Pt RS 27203, Darfield. It is understood that the property owners are in the process of preparing a private plan change, in order to have the subject site rezoned from Rural Outer Plains to Living X zone (or low density residential zone, as reflected in the National Policy Statement).

The test pit and borehole logs, presented in the Appendix of this report, indicate that the subject site is, in general, underlain by soils inferred to be alluvial sediments of late Pleistocene age.

Given the nature, age and consistency of the sediments underlying the subject site, i.e. generally unsaturated very dense sandy gravels, 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.

Based on the results of the investigations and appraisal reported herein, 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, founded in the underlying alluvial sediments.

Foundation design recommendations for future proposed residential development are presented in Sections 7.0 and 8.0 of this report.

The site is, in general, considered suitable for its intended use, with satisfactory conditions for future residential 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 NZS 3604: 2011, New Zealand Standard, Timber Framed Buildings.

# PROPOSED PLAN CHANGE -
# DARFIELD PREFERRED DEVELOPMENT AREA 7
# (MALVERN AREA PLAN)

## MR MERVYN TODD

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## MR MERVYN TODD

## 1.0 INTRODUCTION

This report presents the results of a geotechnical investigation and appraisal undertaken for the site at Lot 4 DP 524058, Pt RS 27204 and Pt RS 27203, Darfield. It is understood that the property owners are in the process of preparing a private plan change, in order to have the subject site rezoned from Rural Outer Plains to Living X zone (or low density residential zone, as reflected in the National Policy Statement).

It is understood this rezoning is to allow subdivision that would involve the creation of lots with an average lot size of not smaller than 650 $m^2$. The proposal will also include provision for a 2 – 3 ha retirement village.

The roughly 70 ha site is bound by Kimberley Road and Horndon Street, located to the west and southeast respectively. Residential properties are situated along the southern boundary of the site, and the properties surrounding the subject site, to the north and east, are rural properties.

The subsurface conditions of the site have been investigated by means of six hand augered boreholes, and twelve machine excavated test pits with associated Dynamic Cone Penetrometer (DCP) scala tests.

A visual appraisal of the site, a study of historical aerial photographs 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 considerations, and to determine the suitability of the subject site in support of an application for rezoning of the land.

## 2.0 AERIAL PHOTOGRAPHS

Historic aerial photographs from 1940 to 2018 were examined, as part of the site appreciation.

Aerial imagery from 1940 indicates that an area in the western part of the site was once covered in trees. The trees are visible in the 1999 aerial photographs. Images from 2009 indicate that the trees were cleared sometime between 1999 and 2009, and this area is now vegetated with paddock grass.

The aerial photographs indicate that the majority of the subject site has been vegetated with paddock grass since at least 1940.

## 3.0 GEOLOGY

In assessing the geology of the site, reference has been made to the Institute of Geological & Nuclear Sciences Geological Map 16, scale 1:250,000, “Christchurch”.

This map indicates that the site is likely to be underlain by “brownish grey river alluvium” of late Pleistocene age.

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

## 4.0 FIELD INVESTIGATION

### 4.1 GENERAL

The field investigation comprised a visual appraisal, twelve machine excavated test pits, numbered TP1 to TP12 inclusive, with associated Dynamic Cone Penetrometer (DCP) tests, and six shallow hand augered boreholes, numbered H1 to H6 inclusive.

The approximate locations of the investigation test positions are shown on Fraser Thomas Ltd drawing G00114-01.

### 4.2 RESULTS OF VISUAL APPRAISAL

A visual appraisal of the subject site was undertaken by a Fraser Thomas Ltd engineering geologist on 1 August 2019.

The site is located on the eastern side of Kimberley Road. Horndon Street is located in close proximity to the south-eastern corner of the subject site. Existing residential properties abut the southern site boundary. The northern and eastern site boundaries abut rural properties.

The topography within the subject site is generally flat, with slight undulations in the land surface, which are likely related to palaeochannels. At the time of the investigation reported herein, the site was generally vegetated with paddock grass and crops.

An existing 3.0 m deep “soak pit” was observed located in the western part of the site. It is understood that this was excavated by the farmer and is used to dispose of overland stormwater from the site.

Two existing ponds, approximately 1.0 m deep, are located along the southern site boundary. It is understood that these ponds are remnant sediment control ponds, which were installed to control sediments generated from the previous subdivisional earthworks, undertaken for the previous subdivision located to the south of the subject site. These ponds were dry at the time of the investigation reported herein.

The approximate inferred locations and extent of the remnant sediment control ponds and the existing soakage pit are shown on the appended drawing G00114-01.

### 4.3 TEST PIT INVESTIGATION

Twelve machine excavated test pits, numbered TP1 to TP12 inclusive, were put down at the site on 1 August 2019, in order to determine the nature and consistency 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 1.7 m and 3.1 m below the ground surface existing at the time of the investigation reported herein (i.e. the existing ground surface).

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

DCP scala tests were carried out at various depths in some of the test pits, in order to determine the density 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 the test pits are shown on drawing G00114-01.

### 4.4 HAND AUGERED BOREHOLE INVESTIGATION

Six hand augered boreholes, numbered H1 to H6 inclusive, were put down at the site on 1 August 2019, in order to determine the nature and consistency of the subsoils underlying the site.

The hand augered boreholes were put down and logged by a qualified Fraser Thomas engineering geologist.

The boreholes were terminated when the soils became too difficult to auger, at depths ranging between approximately 0.3 m and 0.4 m below the existing ground surface.

The logs of the boreholes are presented in Appendix A of this report.

The approximate locations of the hand augered boreholes are shown on drawing G00114-01.

## 5.0 SUBSURFACE CONDITIONS

### 5.1 GENERAL

The test pit and borehole logs, presented in the Appendix of this report, indicate 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 pits and hand augered boreholes 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 silts, was generally encountered at the locations of the test positions, to a depth of between approximately 0.2 m and 0.4 m below the existing ground surface.

A surficial layer of topsoil, approximately 0.6 m thick, was encountered at the location of Test Pit TP1. This thicker layer of topsoil is inferred to be localised and likely associated with previous farm works. This topsoil thickness is not believed to be representative of the topsoil layer thickness across the subject site.

### 5.3 ALLUVIAL SEDIMENTS

An upper layer of soils, generally comprising silts and gravelly silts, inferred to be alluvial sediments of late Pleistocene age, was encountered beneath the surficial layer of topsoil. These sediments were generally encountered to a depth of between approximately 0.4 m and 0.9 m below the existing ground surface, corresponding to a layer thickness of between approximately 0.2 m to 0.4 m.

In situ undrained shear strength values of between approximately 84 kPa and greater than 200 kPa were generally measured in these sediments, using hand held shear vane equipment, corresponding to a stiff to hard consistency.

Soils generally comprising sandy gravels, inferred to be alluvial sediments of late Pleistocene age, were encountered beneath the surficial layers of silts. These sediments were generally encountered to the extent of the machine excavated test pits.

Dynamic Cone Penetrometer (DCP) scala tests undertaken in the sandy gravels generally obtained blow counts of between 4 and 15 blows per 50 mm penetration in these sediments, corresponding to SPT ‘N’ values of greater than 50, generally corresponding to a very dense consistency.

The log of a water bore, put down approximately 50 m to the west of the subject site, has been sourced from Environment Canterbury records.

The existing water bore log indicates that gravels are generally located at shallow depths, which is consistent with the subsoil conditions encountered at the subject site. The bore log indicates that these gravels extend to significant depths beneath the ground surface.

### 5.4 GROUNDWATER

Groundwater was not encountered at the locations of the machine excavated test pits put down at the time of investigation reported herein. Information obtained from water bore logs, located in the vicinity of the site, indicate that the groundwater level in the vicinity of the site is likely to be at depths in excess of 10 m below the ground surface.

## 6.0 LIQUEFACTION POTENTIAL ASSESSMENT

### 6.1 GENERAL

This section of the report presents the results of a site-specific liquefaction potential assessment undertaken for the subject site.

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.

## 6.2 METHOD OF ANALYSIS

Guidelines for the assessment of the liquefaction potential of soils is provided by the New Zealand Geotechnical Society in the document entitled “Geotechnical Earthquake Engineering Practice: Module 1- Guideline for the identification, assessment and mitigation of liquefaction hazards”, dated July 2010.

The July 2010 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 October 2001 report, among others, refers to papers by Youd et al; Seed; Idriss; Boulanger; Robertson and Bray.

The July 2010 guideline suggests a three step process for the liquefaction assessment of sites, being:

(i) Step 1: Assessment of liquefaction susceptibility

(ii) Step 2: Triggering of liquefaction

(iii) Step 3: Consequences of liquefaction

A liquefaction potential assessment of the soils underlying the subject site has been undertaken using the methods suggested by the July 2010 guideline.

### 6.3 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 3.0 of this report, the geological map for the area indicates that the site is likely to be underlain by "brownish grey river alluvium" of late Pleistocene age.

As discussed in Section 5.3 of this report, the results of the field investigations indicate that the site is generally underlain by a surficial layer of silts, which is in turn underlain by sandy gravels. The sandy gravels are generally of a very dense consistency, and are inferred to extend to significant depths below the ground surface.

As discussed in Section 5.4 of this report, the groundwater level in the vicinity of the site is likely to be at depths in excess of 10 m below the ground surface.

Based on the foregoing, given the nature, age and consistency of the sediments underlying the subject site, i.e. generally unsaturated very dense sandy gravels, 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.

## 7.0 FOUNDATION DESIGN CONSIDERATIONS

### 7.1 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 any 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.

### 7.2 THE RISK OF THE SITE BEING ADVERSELY AFFECTED BY GROUND DEFORMATIONS ASSOCIATED WITH LIQUEFACTION

As discussed in Section 6.3 of this report, it is our opinion that the surficial soils underlying the subject 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 low.

Based on the results of the investigations and appraisal reported herein, 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, founded in the underlying alluvial sediments.

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

Fraser Thomas Ltd 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 natural ground.

## 7.3 SHALLOW FOUNDATIONS LOCATED IN CLOSE PROXIMITY TO THE EXISTING SEDIMENT CONTROL PONDS AT THE SITE

As discussed in Section 4.2 of this report, two existing ponds, approximately 1.0 m deep, are located along the southern site boundary. It is understood that these ponds are remnant sediment control ponds, which were installed to control sediments generated from the previous subdivisional earthworks, undertaken for the previous subdivision located to the south of the subject site.

The approximate inferred locations and extent of the remnant sediment control ponds are shown on the appended drawing G00114-01.

Loose sediments are likely to have been deposited in the base of the sediment control ponds.

There is a risk that shallow building foundations founded within the footprint of the existing ponds may be subject to differential settlement.

In order to mitigate the risk of any proposed future shallow foundations being adversely affected by the settlement of sediments in these ponds, it is recommended, unless further specific investigation and appraisal works are undertaken by a Chartered Professional Engineer experienced in geotechnical engineering, that shallow foundations associated with any proposed future dwellings at the site, be located no closer than a horizontal distance of 5 m from the edge of the existing ponds.

## 7.4 FOUNDATIONS LOCATED IN CLOSE PROXIMITY TO THE EXISTING SOAK PIT

As discussed in Section 4.2 of this report, an existing 3.0 m deep “soak pit” was observed located in the western part of the site. It is understood that this was excavated by the farmer and is used to dispose of overland stormwater from the site.

The approximate inferred location and extent of the existing soak pit is shown on the appended drawing G00114-01.

There is, in our opinion, a risk that shallow foundations founded within the vicinity of the soak pit, may be subject to differential settlement, which may adversely affect future building development in this area. It is therefore recommended that further site specific geotechnical investigation works be undertaken, for any proposed building development located in the vicinity of the existing soak pit, in order to provide appropriate recommendations and parameters for foundation design purposes.

# 8.0 ALLOWABLE FOUNDATION BEARING PRESSURES

## 8.1 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.

## 8.2 SHALLOW PAD OR STRIP FOOTINGS

A minimum ultimate static bearing capacity value for vertical loading of 300 kPa is recommended for shallow pad or strip footings founded within the underlying alluvial sediments. It is recommended that a strength reduction factor ($\Phi_{bc}$) 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 pad or strip footings, founded within the underlying alluvial sediments.

**TABLE 1: ALLOWABLE FOUNDATION BEARING PRESSURES FOR SHALLOW PAD OR STRIP FOOTINGS WITHIN 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 |

# 9.0 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 any new 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 dwelling 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 dwelling 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.

## 10.0 STORMWATER AND EFFLUENT DISPOSAL

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

## 11.0 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.

## 12.0 CONCLUSIONS AND RECOMMENDATIONS

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

### 12.1 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) The site is, in general, considered suitable for its intended use, with satisfactory conditions for future residential 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 NZS 3604: 2011, New Zealand Standard, Timber Framed Buildings.

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 pits and hand augered boreholes 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 considerations, and to determine the suitability of the subject site in support of an application for rezoning of the land.

(c) The test pit and borehole logs, presented in the Appendix of this report, indicate that the subject site is, in general, underlain by soils inferred to be alluvial sediments of late Pleistocene age.

(d) A surficial layer of topsoil, generally comprising silts, was generally encountered at the locations of the test positions, to a depth of between approximately 0.2 m and 0.4 m below the existing ground surface.

A surficial layer of topsoil, approximately 0.6 m thick, was encountered at the location of Test Pit TP1. This thicker layer of topsoil is inferred to be localised and likely associated with previous farm works. This topsoil thickness is not believed to be representative of the topsoil layer thickness across the subject site.

(e) An upper layer of soils, generally comprising stiff to hard silts and gravelly silts, inferred to be alluvial sediments of late Pleistocene age, was encountered beneath the surficial layer of topsoil. These sediments were generally encountered to a depth of between approximately 0.4 m and 0.9 m below the existing ground surface, corresponding to a layer thickness of between approximately 0.2 m to 0.4 m.

(f) Soils generally comprising very dense sandy gravels, inferred to be alluvial sediments of late Pleistocene age, were encountered beneath the surficial layers of silts. These sediments were generally encountered to the extent of the machine excavated test pits.

The log of a water bore, put down approximately 50 m to the west of the subject site, has been sourced from Environment Canterbury records.

The existing water bore log indicates that gravels are generally located at shallow depths, which is consistent with the subsoil conditions encountered at the subject site. The bore log indicates that these gravels extend to significant depths beneath the ground surface.

(g) Groundwater was not encountered at the locations of the machine excavated test pits put down at the time of investigation reported herein. Information obtained from water bore logs, located in the vicinity of the site, indicate that the groundwater level in the vicinity of the site is likely to be at depths in excess of 10 m below the ground surface.

(h) Given the nature, age and consistency of the sediments underlying the subject site, i.e. generally unsaturated very dense sandy gravels, 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.

(i) Based on the results of the investigations and appraisal reported herein, 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, founded in the underlying alluvial sediments.

(j) 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 any 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.

## 12.2 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) That any proposed shallow foundations be founded beneath the surficial topsoil into the underlying competent alluvial sediments.

Fraser Thomas Ltd 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 natural ground.

(b) That, unless further specific investigation and appraisal works are undertaken by a Chartered Professional Engineer experienced in geotechnical engineering, shallow foundations associated with any proposed future dwellings at the site, should be located no closer than a horizontal distance of 5 m from the edge of the existing ponds.

(c) That further site specific geotechnical investigation works be undertaken, for any proposed building development located in the vicinity of the existing soak pit, in order to provide appropriate recommendations and parameters for foundation design purposes.

(d) A minimum ultimate static bearing capacity value for vertical loading of 300 kPa is recommended for shallow pad or strip footings founded within the underlying alluvial sediments. It is recommended that a strength reduction factor ($\Phi_{bc}$) 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.

(e) That the location and depth of any buried services should be verified at the site prior to the commencement of any new foundation construction.

(f) That, if any foundations of any proposed new dwelling 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 dwelling be designed to span across the trench backfill and the adjacent zone of influence.

(g) 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) 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.

## 13.0 LIMITATIONS

The professional opinion expressed herein has been prepared solely for, and is furnished to our client, Mr Mervyn Todd and his professional advisors, and Selwyn 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.

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.**

**K E TWOHILL**
Engineering Geologist

**Report reviewed and approved by:**

**M V REED**
Director
Chartered Professional Engineer

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# Appendix A

# Field Investigation Results

# *Hand Augered Boreholes*

# 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

| | | | |
|---|---|---|---|
| TS | TOPSOIL | ▨ | COBBLES |
| ▤ | CLAY | ▨ | BOULDERS |
| ▨ | SILT | ▨ | PEAT |
| ▨ | SAND | ▩ | FILL |
| ▨ | GRAVEL | | |

## 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 | ▨ | RYHOLITE |
| ▤ | MUDSTONE | ▨ | ANDESITE |
| ▨ | SANDSTONE | ▨ | BASALT |
| ▨ | CONGLOMERATE | | |
| ▲ | BRECCIA | | |

## 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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# HAND AUGER LOG

**Hole No:** H1

| Project No: | Project: | Shear Vane: | Date Drilled: | Logged By: | Checked By: |
|---|---|---|---|---|---|
| CH00114 | Mervyn Todd<br>Kimberley Road, Darfield, Canterbury | | 01/08/2019 | KT | |

Undrained Shear Strength (kPa): Vane readings corrected as per BS 1377 — ● Shear Vane ○ Residual Shear Vane — 50, 100, 150, 200 — Values

Dynamic Cone Penetrometer: Test Method: NZS 4402:1988, Test 6.5.2 (Blows / 0mm) — 2, 4, 6, 8, 10, 12, 14, 16

| Depth (m) | Description of Strata | Geological Unit | Graphic Log | Undrained Shear Strength (kPa) Values | Groundwater |
|---|---|---|---|---|---|
| 0.0–0.3 | SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S | TS | | GWNE |
| 0.3–0.4 | SILT, gravelly (fine to medium, subangular to subrounded), yellowish brown, stiff, moist, non plastic [ALLUVIAL SEDIMENTS] | | | UTP | |

EOH: 0.40 m TOO DIFFICULT TO AUGER

Depth scale (m): 0.2, 0.4, 0.6, 0.8, 1.0, 1.2, 1.4, 1.6, 1.8, 2.0, 2.2, 2.4, 2.6, 2.8, 3.0, 3.2, 3.4, 3.6, 3.8, 4.0, 4.2, 4.4, 4.6, 4.8

**Remarks:**

**Datum:**

**Coordinates:**

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# HAND AUGER LOG

**Hole No:** H2

| **Project No:** | **Project:** | **Shear Vane:** | **Date Drilled:** | **Logged By:** | **Checked By:** |
|---|---|---|---|---|---|
| CH00114 | Mervyn Todd<br>Kimberley Road, Darfield, Canterbury | | 01/08/2019 | KT | |

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 / 0mm)

| Depth (m) | Description of Strata | Geological Unit | Graphic Log | Undrained Shear Strength (kPa) 50 100 150 200 | Values | Depth (m) | Dynamic Cone Penetrometer 2 4 6 8 10 12 14 16 | Groundwater |
|---|---|---|---|---|---|---|---|---|
| 0.0 – 0.3 | SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S | TS | | | | | GWNE |
| 0.3 | EOH: 0.30 m TOO DIFFICULT TO AUGER | | | | | | | |

Depth scale (m): 0.2, 0.4, 0.6, 0.8, 1.0, 1.2, 1.4, 1.6, 1.8, 2.0, 2.2, 2.4, 2.6, 2.8, 3.0, 3.2, 3.4, 3.6, 3.8, 4.0, 4.2, 4.4, 4.6, 4.8

**Remarks:**

**Datum:**

**Coordinates:**

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# HAND AUGER LOG

**Hole No:** **H3**

| Project No: | Project: | Shear Vane: | Date Drilled: | Logged By: | Checked By: |
|---|---|---|---|---|---|
| CH00114 | Mervyn Todd<br>Kimberley Road, Darfield, Canterbury | | 01/08/2019 | KT | |

Undrained Shear Strength (kPa): Vane readings corrected as per BS 1377 — ● Shear Vane ○ Residual Shear Vane — 50 100 150 200 Values

Dynamic Cone Penetrometer: Test Method: NZS 4402:1988, Test 6.5.2 (Blows / 0mm) — 2 4 6 8 10 12 14 16

| Depth (m) | Description of Strata | Geological Unit | Graphic Log | Undrained Shear Strength (kPa) | Depth (m) | Dynamic Cone Penetrometer | Groundwater |
|---|---|---|---|---|---|---|---|
| 0.2 | SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S | | | 0.2 | | GWNE |
| 0.4 | EOH: 0.30 m TOO DIFFICULT TO AUGER | | | | 0.4 | | |
| 0.6 | | | | | 0.6 | | |
| 0.8 | | | | | 0.8 | | |
| **1.0** | | | | | **1.0** | | |
| 1.2 | | | | | 1.2 | | |
| 1.4 | | | | | 1.4 | | |
| 1.6 | | | | | 1.6 | | |
| 1.8 | | | | | 1.8 | | |
| **2.0** | | | | | **2.0** | | |
| 2.2 | | | | | 2.2 | | |
| 2.4 | | | | | 2.4 | | |
| 2.6 | | | | | 2.6 | | |
| 2.8 | | | | | 2.8 | | |
| **3.0** | | | | | **3.0** | | |
| 3.2 | | | | | 3.2 | | |
| 3.4 | | | | | 3.4 | | |
| 3.6 | | | | | 3.6 | | |
| 3.8 | | | | | 3.8 | | |
| **4.0** | | | | | **4.0** | | |
| 4.2 | | | | | 4.2 | | |
| 4.4 | | | | | 4.4 | | |
| 4.6 | | | | | 4.6 | | |
| 4.8 | | | | | 4.8 | | |

**Remarks:**

**Datum:**

**Coordinates:**

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# HAND AUGER LOG

**Hole No:** H4

| **Project No:** | **Project:** | **Shear Vane:** | **Date Drilled:** | **Logged By:** | **Checked By:** |
|---|---|---|---|---|---|
| CH00114 | Mervyn Todd<br>Kimberley Road, Darfield, Canterbury | 2512 | 01/08/2019 | KT | |

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 / 0mm)

| Depth (m) | Description of Strata | Geological Unit | Graphic Log | Undrained Shear Strength (kPa) Values | Dynamic Cone Penetrometer | Groundwater |
|---|---|---|---|---|---|---|
| | SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S | TS | | | GWNE |
| | SILT, gravelly (fine to medium, subangular to subrounded), yellowish brown, stiff, moist, non plastic [ALLUVIAL SEDIMENTS] | | | 84 | | |

EOH: 0.40 m TOO DIFFICULT TO AUGER

**Remarks:**

**Datum:**

**Coordinates:**

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# HAND AUGER LOG

**Hole No:** H5

| **Project No:** | **Project:** | **Shear Vane:** | **Date Drilled:** | **Logged By:** | **Checked By:** |
|---|---|---|---|---|---|
| CH00114 | Mervyn Todd<br>Kimberley Road, Darfield, Canterbury | | 01/08/2019 | KT | |

**Undrained Shear Strength (kPa)**: Vane readings corrected as per BS 1377; ● Shear Vane ○ Residual Shear Vane; 50, 100, 150, 200, Values

**Dynamic Cone Penetrometer**: Test Method: NZS 4402:1988, Test 6.5.2 (Blows / 0mm); 2, 4, 6, 8, 10, 12, 14, 16

| Depth (m) | Description of Strata | Geological Unit | Graphic Log | Undrained Shear Strength (kPa) | Depth (m) | Dynamic Cone Penetrometer | Groundwater |
|---|---|---|---|---|---|---|---|
| 0.2 | SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S | TS | | 0.2 | | GWNE |
| 0.4 | EOH: 0.30 m TOO DIFFICULT TO AUGER | | | | 0.4 | | |
| 0.6 | | | | | 0.6 | | |
| 0.8 | | | | | 0.8 | | |
| **1.0** | | | | | **1.0** | | |
| 1.2 | | | | | 1.2 | | |
| 1.4 | | | | | 1.4 | | |
| 1.6 | | | | | 1.6 | | |
| 1.8 | | | | | 1.8 | | |
| **2.0** | | | | | **2.0** | | |
| 2.2 | | | | | 2.2 | | |
| 2.4 | | | | | 2.4 | | |
| 2.6 | | | | | 2.6 | | |
| 2.8 | | | | | 2.8 | | |
| **3.0** | | | | | **3.0** | | |
| 3.2 | | | | | 3.2 | | |
| 3.4 | | | | | 3.4 | | |
| 3.6 | | | | | 3.6 | | |
| 3.8 | | | | | 3.8 | | |
| **4.0** | | | | | **4.0** | | |
| 4.2 | | | | | 4.2 | | |
| 4.4 | | | | | 4.4 | | |
| 4.6 | | | | | 4.6 | | |
| 4.8 | | | | | 4.8 | | |

**Remarks:**

**Datum:**

**Coordinates:**

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# HAND AUGER LOG

**Hole No:**

**H6**

| **Project No:** | **Project:** | **Shear Vane:** | **Date Drilled:** | **Logged By:** | **Checked By:** |
|---|---|---|---|---|---|
| **CH00114** | **Mervyn Todd**<br>**Kimberley Road, Darfield, Canterbury** | | 01/08/2019 | KT | |

| **Depth (m)** | **Description of Strata** | **Geological Unit** | **Graphic Log** | **Undrained Shear Strength (kPa)** Vane readings corrected as per BS 1377 ● Shear Vane ○ Residual Shear Vane 50 100 150 200 | Values | **Depth (m)** | **Dynamic Cone Penetrometer** Test Method: NZS 4402:1988, Test 6.5.2 (Blows / 0mm) 2 4 6 8 10 12 14 16 | **Groundwater** |
|---|---|---|---|---|---|---|---|---|
| 0.0–0.30 | SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S | TS | | | | | GWNE |
| 0.30 | EOH: 0.30 m TOO DIFFICULT TO AUGER | | | | | | | |

Depth (m): 0.2, 0.4, 0.6, 0.8, **1.0**, 1.2, 1.4, 1.6, 1.8, **2.0**, 2.2, 2.4, 2.6, 2.8, **3.0**, 3.2, 3.4, 3.6, 3.8, **4.0**, 4.2, 4.4, 4.6, 4.8

**Remarks:**

**Datum:**

**Coordinates:**

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# *Machine Excavated Test Pits*

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# HAND AUGER LOG

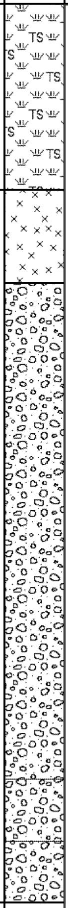
**Hole No:** **TP1**

| Project No: | Project: | Shear Vane: | Date Drilled: | Logged By: | Checked By: |
|---|---|---|---|---|---|
| CH00114 | Mervyn Todd<br>Kimberley Road, Darfield, Canterbury | 2512 | 01/08/2019 | KT | |

**Undrained Shear Strength (kPa)**
Vane readings corrected as per BS 1377
● Shear Vane ○ Residual Shear Vane
50 100 150 200 Values

**Dynamic Cone Penetrometer**
Test Method: NZS 4402:1988, Test 6.5.2
(Blows / 50mm)
2 4 6 8 10 12 14 16

| Depth (m) | Description of Strata | Geological Unit | Values | Groundwater |
|---|---|---|---|---|
| 0.0 - 0.6 | SILT, some gravel, dark brown, moist, rootlets [TOPSOIL]<br>0.3 m: becomes gravelly | T/S | | GWNE |
| 0.6 - 0.9 | SILT, some sand (fine), trace gravel (fine, subangular), yellowish brown, hard, moist [ALLUVIAL SEDIMENTS] | River Alluvium | UTP | |
| 0.9 - 2.9 | GRAVEL (fine to coarse, subrounded, greywacke), sandy (fine to coarse), trace cobbles, greyish brown, very dense, moist<br>2.5 m - 2.7 m: Colour change to light grey, less fines | | | |
| 2.9 | EOH: 2.90 m TARGET DEPTH | | | |



**Remarks:**

**Datum:**

**Coordinates:**

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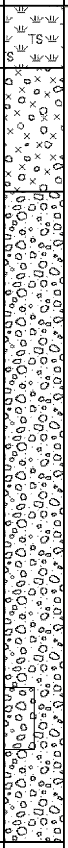
# HAND AUGER LOG

**Hole No:** **TP2**

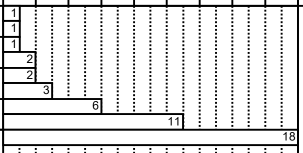
| Project No: | Project: | Shear Vane: | Date Drilled: | Logged By: | Checked By: |
|---|---|---|---|---|---|
| **CH00114** | **Mervyn Todd**<br>**Kimberley Road, Darfield, Canterbury** | 2512 | 01/08/2019 | KT | |

| Depth (m) | Description of Strata | Geological Unit | Undrained Shear Strength (kPa) Values | Groundwater |
|---|---|---|---|---|
| 0.0 – 0.2 | SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S | | GWNE |
| 0.2 – 0.6 | SILT, gravelly (fine, subangular), minor sand (fine), yellowish brown, hard, moist, low plasticity [ALLUVIAL SEDIMENTS] | River Alluvium | UTP (0.4) | |
| 0.6 – 2.7 | GRAVEL (fine to coarse, subrounded, greywacke), sandy (fine to coarse), minor cobbles, trace boulders, greyish brown, very dense, moist<br>2.2 m - 2.4 m: Lense of GRAVEL (fine to medium), light grey, wet | River Alluvium | | |
| 2.70 | EOH: 2.70 m TARGET DEPTH | | | |

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)



| | |
|---|---|
| 0.00 – 0.05 | 1 |
| 0.05 – 0.10 | 1 |
| 0.10 – 0.15 | 1 |
| 0.15 – 0.20 | 2 |
| 0.20 – 0.25 | 2 |
| 0.25 – 0.30 | 3 |
| 0.30 – 0.35 | 6 |
| 0.35 – 0.40 | 11 |
| 0.40 – 0.45 | 18 |

**Remarks:**

**Datum:**

**Coordinates:**

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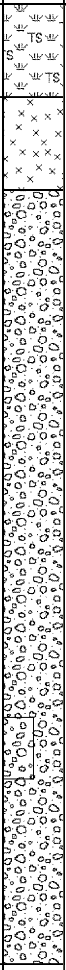
# HAND AUGER LOG

**Hole No:** **TP3**

| **Project No:** | **Project:** | **Shear Vane:** | **Date Drilled:** | **Logged By:** | **Checked By:** |
|---|---|---|---|---|---|
| **CH00114** | **Mervyn Todd**<br>**Kimberley Road, Darfield, Canterbury** | 2512 | 01/08/2019 | KT | |

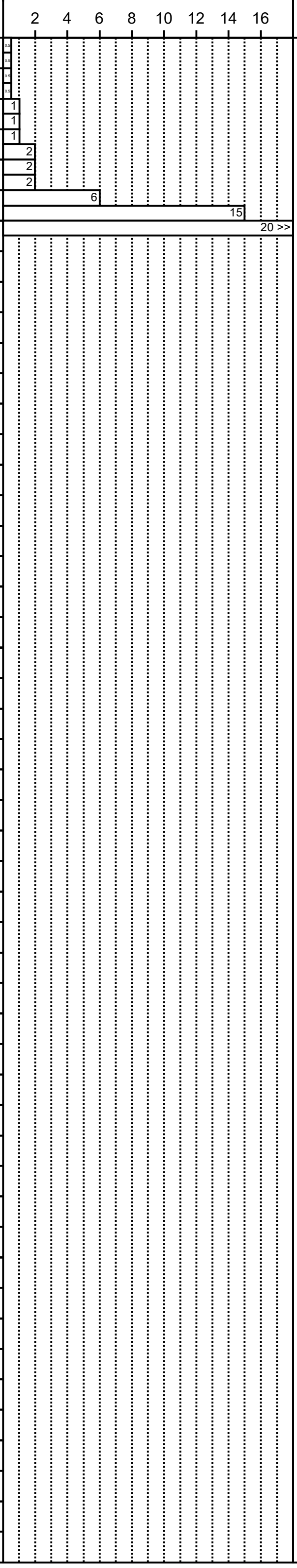
| **Depth (m)** | **Description of Strata** | **Geological Unit** | **Undrained Shear Strength (kPa)** Values | **Groundwater** |
|---|---|---|---|---|
| 0.0 – 0.3 | SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S | UTP | GWNE |
| 0.3 – 0.65 | SILT, gravelly (fine to medium, subangular to subrounded), minor sand (fine), yellowish brown, hard, moist, low plasticity [ALLUVIAL SEDIMENTS] | River Alluvium | | |
| 0.65 – 3.10 | GRAVEL (fine to coarse, subrounded, greywacke), sandy (fine to coarse), trace cobbles, trace boulders, greyish brown, very dense, moist | River Alluvium | | |
| | 2.3 m - 2.5 m: Lense of GRAVEL (fine to medium), grey, wet | | | |
| 3.10 | EOH: 3.10 m TARGET DEPTH | | | |

**Graphic Log**



**Undrained Shear Strength (kPa)**
Vane readings corrected as per BS 1377
● Shear Vane ○ Residual Shear Vane
50 100 150 200 Values

**Dynamic Cone Penetrometer**
Test Method: NZS 4402:1988, Test 6.5.2
(Blows / 50mm)



**Remarks:**

**Datum:**

**Coordinates:**

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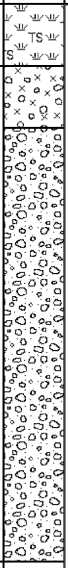
# HAND AUGER LOG

**Hole No:** TP4

| Project No: | Project: | Shear Vane: | Date Drilled: | Logged By: | Checked By: |
|---|---|---|---|---|---|
| CH00114 | Mervyn Todd<br>Kimberley Road, Darfield, Canterbury | | 01/08/2019 | KT | |

| Depth (m) | Description of Strata | Geological Unit | Groundwater |
|---|---|---|---|
| 0.0 – 0.2 | SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S | GWNE |
| 0.2 – 0.5 | SILT, gravelly (fine to medium, subangular to subrounded), minor sand (fine), yellowish brown, hard, moist, non plastic [ALLUVIAL | River Alluvium | |
| 0.5 – 1.8 | GRAVEL (fine to coarse, subrounded, greywacke), sandy (fine to coarse), with cobbles, greyish brown, very dense, moist | | |
| 1.8 | EOH: 1.80 m TARGET DEPTH | | |



Undrained Shear Strength (kPa)
Vane readings corrected as per BS 1377
● Shear Vane ○ Residual Shear Vane
50 100 150 200 Values

Dynamic Cone Penetrometer
Test Method: NZS 4402:1988, Test 6.5.2
(Blows / 50mm)
2 4 6 8 10 12 14 16

Depth (m): 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 2.0 2.2 2.4 2.6 2.8 3.0 3.2 3.4 3.6 3.8 4.0 4.2 4.4 4.6 4.8

**Remarks:**

**Datum:**

**Coordinates:**

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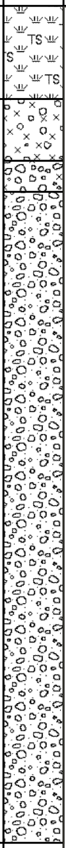
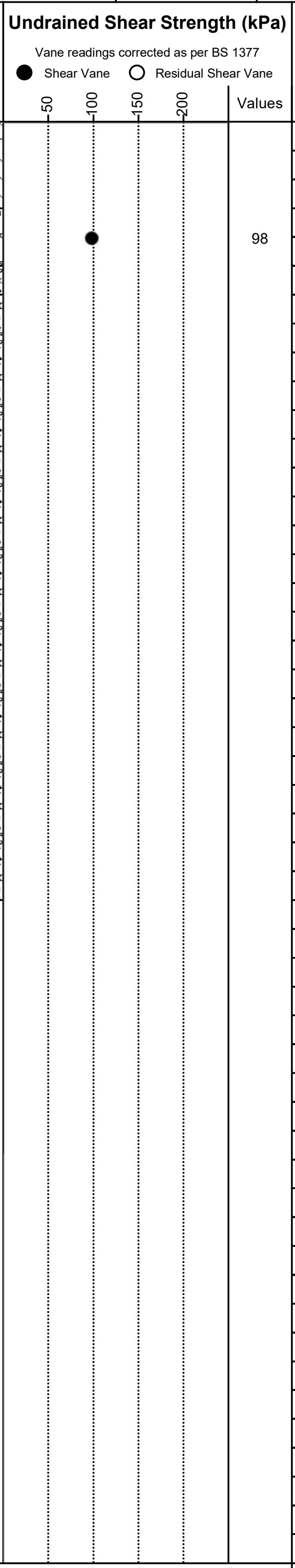
# HAND AUGER LOG

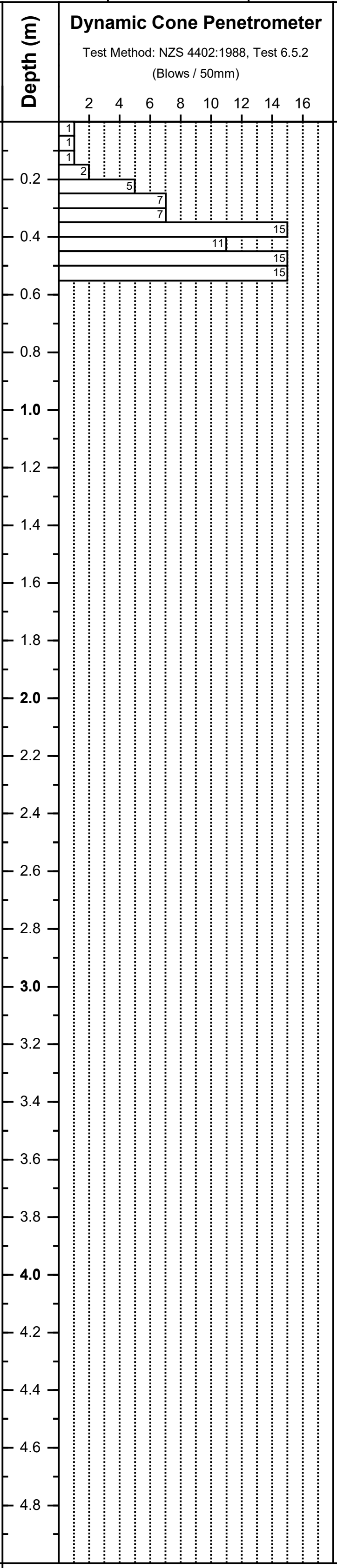
**Hole No:** TP5

| **Project No:** | **Project:** | **Shear Vane:** | **Date Drilled:** | **Logged By:** | **Checked By:** |
|---|---|---|---|---|---|
| CH00114 | Mervyn Todd<br>Kimberley Road, Darfield, Canterbury | 2512 | 01/08/2019 | KT | |

| Description of Strata | Geological Unit |
|---|---|
| SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S |
| SILT, gravelly (fine to medium, subangular to subrounded), minor sand (fine), yellowish brown, stiff, moist, non plastic [ALLUVIAL | River Alluvium |
| GRAVEL (fine to medium, subrounded, greywacke), silty, yellowish brown, very dense, moist | River Alluvium |
| GRAVEL (fine to coarse, subrounded, greywacke), sandy (fine to coarse), trace cobbles, greyish brown, very dense, moist | River Alluvium |
| EOH: 2.80 m TARGET DEPTH | |







**Groundwater:** GWNE

**Remarks:**

**Datum:**

**Coordinates:**

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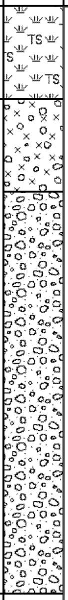
# HAND AUGER LOG

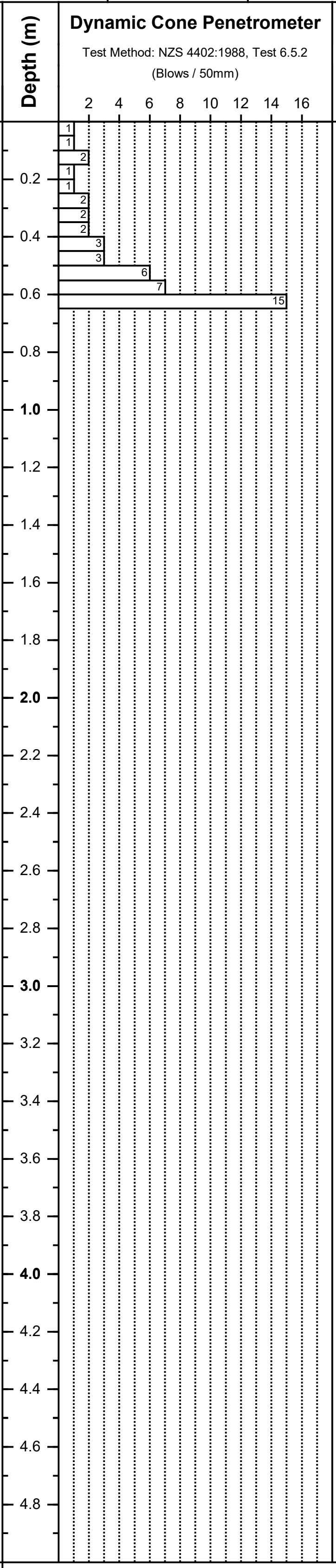
**Hole No:** TP6

| Project No: | Project: | Shear Vane: | Date Drilled: | Logged By: | Checked By: |
|---|---|---|---|---|---|
| CH00114 | Mervyn Todd<br>Kimberley Road, Darfield, Canterbury | 2512 | 01/08/2019 | KT | |

| Description of Strata | Geological Unit | Undrained Shear Strength (kPa) Values | Groundwater |
|---|---|---|---|
| SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S | | GWNE |
| SILT, gravelly (fine to medium, subangular to subrounded), minor sand (fine), yellowish brown, hard, moist, non plastic, trace rootlets [ALLUVIAL SEDIMENTS] | River Alluvium | UTP | |
| GRAVEL (fine to coarse, subrounded, greywacke), sandy (fine to coarse), greyish brown, very dense, moist | | | |
| EOH: 1.90 m TARGET DEPTH | | | |



Undrained Shear Strength (kPa)
Vane readings corrected as per BS 1377
● Shear Vane ○ Residual Shear Vane



**Remarks:**

**Datum:**

**Coordinates:**

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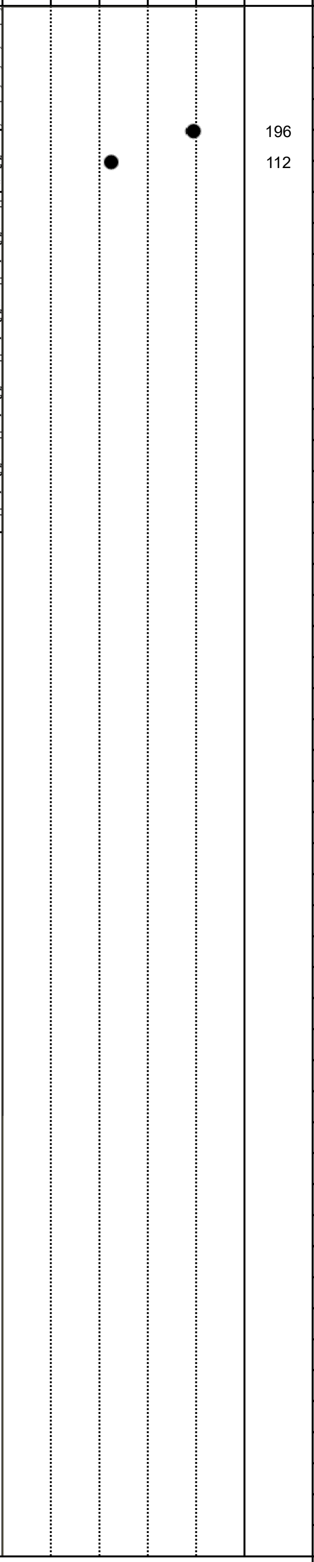
# HAND AUGER LOG

**Hole No:** **TP7**

| **Project No:** | **Project:** | **Shear Vane:** | **Date Drilled:** | **Logged By:** | **Checked By:** |
|---|---|---|---|---|---|
| CH00114 | Mervyn Todd<br>Kimberley Road, Darfield, Canterbury | 2512 | 01/08/2019 | KT | |

| Depth (m) | Description of Strata | Geological Unit | Undrained Shear Strength (kPa) Values | Dynamic Cone Penetrometer | Groundwater |
|---|---|---|---|---|---|
| 0.0 – 0.4 | SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S | | | GWNE |
| 0.4 – 0.8 | SILT, gravelly (fine to medium, subangular to subrounded), minor sand (fine), yellowish brown, very stiff, moist, non plastic [ALLUVIAL SEDIMENTS] | River Alluvium | 196<br>112 | | |
| 0.8 – 1.7 | GRAVEL (fine to coarse, subrounded, greywacke), sandy (fine to coarse), minor cobbles, greyish brown, very dense, moist | River Alluvium | | | |
| 1.70 | EOH: 1.70 m TARGET DEPTH | | | | |

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)

**Remarks:**

**Datum:**

**Coordinates:**

Generated with CORE-GS by Geroc - Hand Auger MASTER - 9/09/2019 8:59:43 AM

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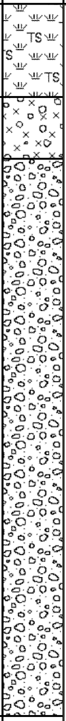
# HAND AUGER LOG

**Hole No:** **TP8**

| **Project No:** | **Project:** | **Shear Vane:** | **Date Drilled:** | **Logged By:** | **Checked By:** |
|---|---|---|---|---|---|
| **CH00114** | **Mervyn Todd**<br>**Kimberley Road, Darfield, Canterbury** |  | 01/08/2019 | KT |  |

| Description of Strata | Geological Unit | Groundwater |
|---|---|---|
| SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S | GWNE |
| SILT, gravelly (fine to medium, subangular to subrounded), yellowish brown, hard, moist, non plastic [ALLUVIAL SEDIMENTS] | River Alluvium |  |
| GRAVEL (fine to coarse, subrounded, greywacke), sandy (fine to coarse), trace cobbles, greyish brown, dense to very dense, moist |  |  |
| EOH: 2.30 m TARGET DEPTH |  |  |

**Graphic Log**



**Undrained Shear Strength (kPa)**

Vane readings corrected as per BS 1377

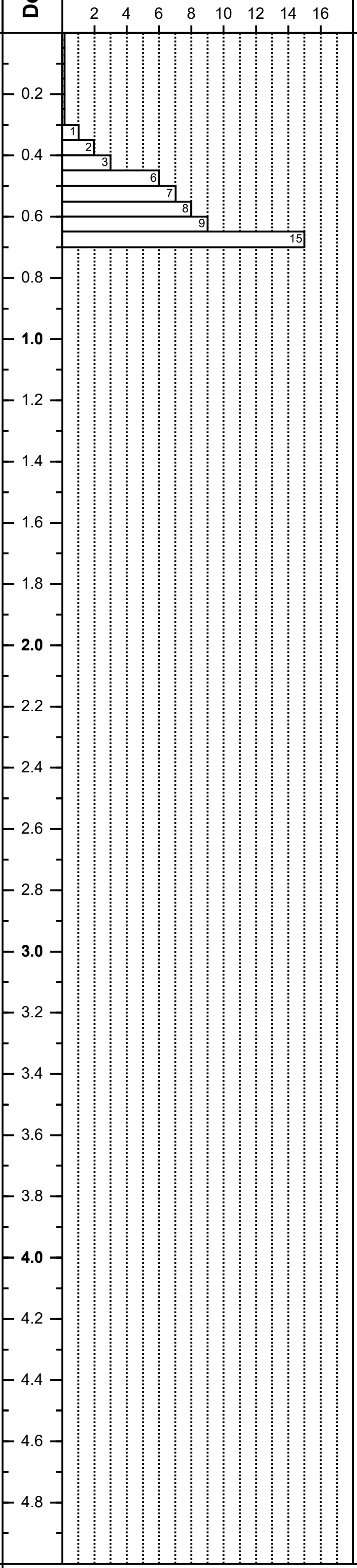
● Shear Vane ○ Residual Shear Vane

50 100 150 200 Values

**Dynamic Cone Penetrometer**

Test Method: NZS 4402:1988, Test 6.5.2

(Blows / 50mm)



**Remarks:**

**Datum:**

**Coordinates:**

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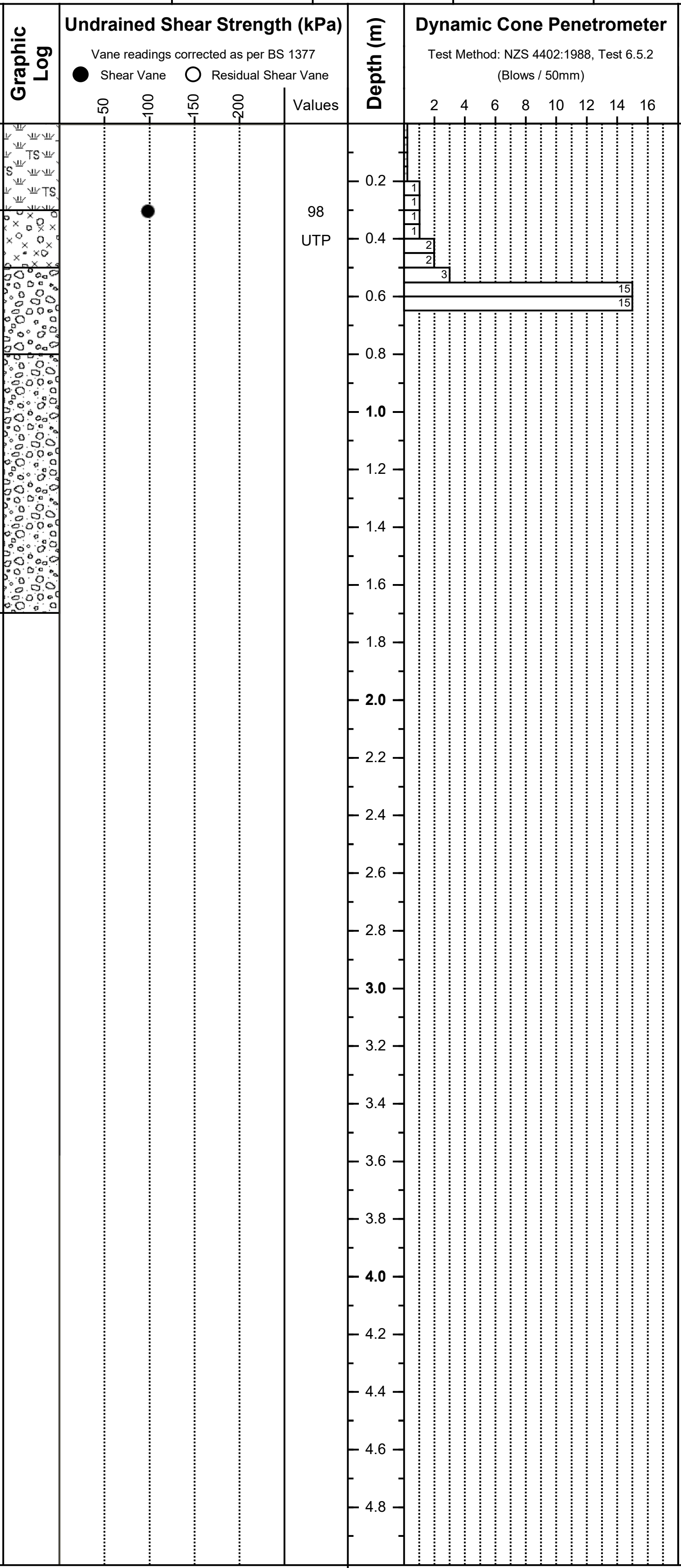
# HAND AUGER LOG

**Hole No:** TP9

| Project No: | Project: | Shear Vane: | Date Drilled: | Logged By: | Checked By: |
|---|---|---|---|---|---|
| CH00114 | Mervyn Todd<br>Kimberley Road, Darfield, Canterbury | 2512 | 01/08/2019 | KT | |

| Depth (m) | Description of Strata | Geological Unit | Groundwater |
|---|---|---|---|
| 0.0 – 0.3 | SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S | GWNE |
| 0.3 – 0.6 | SILT, gravelly (fine to medium, subangular to subrounded), yellowish brown, stiff to hard, moist, non plastic [ALLUVIAL SEDIMENTS] | River Alluvium | |
| 0.6 – 0.9 | GRAVEL (fine to medium, subrounded, greywacke), sandy (fine to coarse), greyish brown, very dense, moist | River Alluvium | |
| 0.9 – 1.7 | GRAVEL (fine to coarse, subrounded, greywacke), sandy (fine to coarse), trace cobbles, trace boulders, greyish brown, very dense, moist | River Alluvium | |

EOH: 1.70 m TARGET DEPTH



**Remarks:**

**Datum:**

**Coordinates:**

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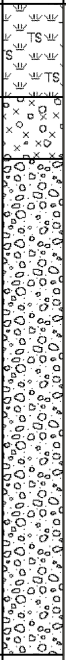
# HAND AUGER LOG

**Hole No:** **TP10**

| Project No: | Project: | Shear Vane: | Date Drilled: | Logged By: | Checked By: |
|---|---|---|---|---|---|
| CH00114 | Mervyn Todd<br>Kimberley Road, Darfield, Canterbury | | 01/08/2019 | KT | |

| Depth (m) | Description of Strata | Geological Unit | Graphic Log | Undrained Shear Strength (kPa)<br>Vane readings corrected as per BS 1377<br>● Shear Vane ○ Residual Shear Vane<br>50 100 150 200 Values | Depth (m) | Dynamic Cone Penetrometer<br>Test Method: NZS 4402:1988, Test 6.5.2<br>(Blows / 50mm)<br>2 4 6 8 10 12 14 16 | Groundwater |
|---|---|---|---|---|---|---|---|
| | SILT, some gravel, dark brown, moist, rootlets<br>[TOPSOIL] | T/S |  | | | | GWNE |
| | SILT, gravelly (fine to medium, subangular to subrounded), minor sand (fine), yellowish brown, moist, non plastic<br>[ALLUVIAL SEDIMENTS] | River Alluvium | | | | | |
| | GRAVEL (fine to coarse, subrounded, greywacke), sandy (fine to coarse), trace cobbles, greyish brown, very dense, moist | | | | | | |
| | EOH: 2.10 m TARGET DEPTH | | | | | | |

Depth (m): 0.2, 0.4, 0.6, 0.8, **1.0**, 1.2, 1.4, 1.6, 1.8, **2.0**, 2.2, 2.4, 2.6, 2.8, **3.0**, 3.2, 3.4, 3.6, 3.8, **4.0**, 4.2, 4.4, 4.6, 4.8

**Remarks:**

**Datum:**

**Coordinates:**

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# HAND AUGER LOG

**Hole No:** TP11

| **Project No:** | **Project:** | **Shear Vane:** | **Date Drilled:** | **Logged By:** | **Checked By:** |
|---|---|---|---|---|---|
| CH00114 | Mervyn Todd<br>Kimberley Road, Darfield, Canterbury | | 01/08/2019 | KT | |

| Description of Strata | Geological Unit | Groundwater |
|---|---|---|
| SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S | GWNE |
| GRAVEL (fine to medium, subangular to subrounded), silty, yellowish brown, medium dense to dense, moist [ALLUVIAL SEDIMENTS] | River Alluvium | |
| GRAVEL (fine to coarse, subrounded, greywacke), sandy (fine to coarse), trace cobbles, greyish brown, very dense, moist | | |
| EOH: 2.10 m TARGET DEPTH | | |

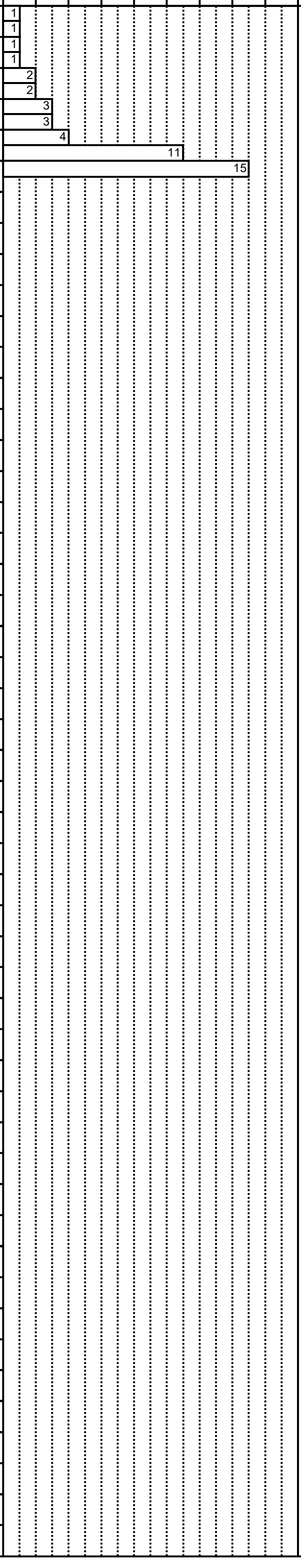
**Undrained Shear Strength (kPa)**

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)



**Remarks:**

**Datum:**

**Coordinates:**

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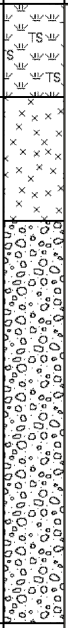
# HAND AUGER LOG

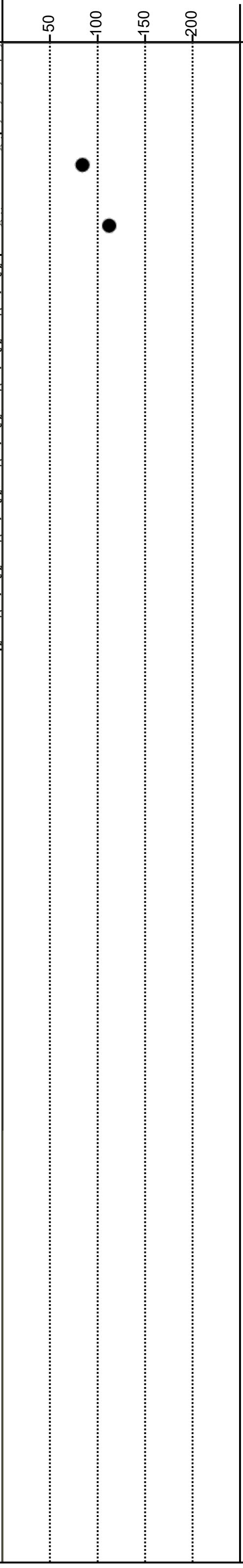
**Hole No:**
# TP12

| Project No: | Project: | Shear Vane: | Date Drilled: | Logged By: | Checked By: |
|---|---|---|---|---|---|
| **CH00114** | **Mervyn Todd**<br>**Kimberley Road, Darfield, Canterbury** | 2512 | 01/08/2019 | KT | |

| Depth (m) | Description of Strata | Geological Unit | Undrained Shear Strength (kPa) Values | Groundwater |
|---|---|---|---|---|
| | SILT, some gravel, dark brown, moist, rootlets [TOPSOIL] | T/S | | GWNE |
| 0.4 | SILT, some gravel (fine to medium, subangular to subrounded), yellowish brown, stiff to very stiff, moist, non plastic [ALLUVIAL SEDIMENTS] | River Alluvium | 84 | |
| 0.6 | | | 112 | |
| | GRAVEL (fine to coarse, subrounded, greywacke), sandy (fine to coarse), trace cobbles, greyish brown, moist | | | |
| 2.0 | EOH: 2.00 m TARGET DEPTH | | | |



**Undrained Shear Strength (kPa)**
Vane readings corrected as per BS 1377
● Shear Vane ○ Residual Shear Vane



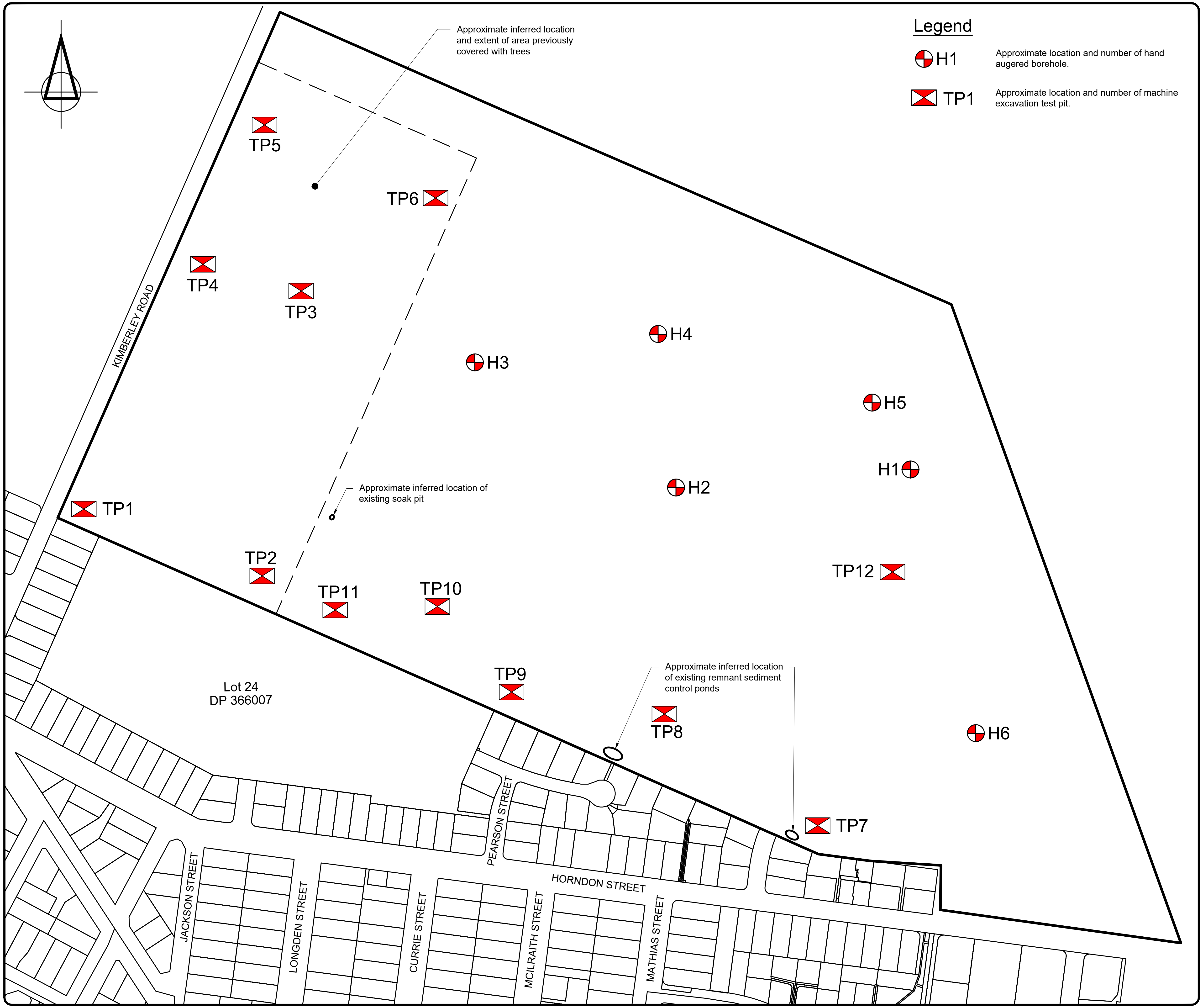
**Dynamic Cone Penetrometer**
Test Method: NZS 4402:1988, Test 6.5.2
(Blows / 50mm)
2 4 6 8 10 12 14 16

**Remarks:**

**Datum:**

**Coordinates:**

Generated with CORE-GS by Geroc - Hand Auger MASTER - 9/09/2019 8:59:21 AM



| | | | APPROVED | DATE |
|---|---|---|---|---|
| SURVEYED | | | MVR | |
| DESIGNED | | | | |
| DRAWN | KT | 08/07/19 | | |
| CAD | G SINGH | 10/07/19 | | |
| CHECKED | | | | |

| REVISION | CHANGES | CHECKED | DATE |
|---|---|---|---|

NOTES

1. This plan has been adopted from Quick map. The location and extent of the site boundaries and site features are therefore considered to be approximate only.

CLIENT

MR MERVYN TODD

PROJECT

KIMBERLEY ROAD
DARFIELD

TITLE

SITE PLAN

Fraser Thomas

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TEL+64-3-358 5936

SCALE 1:4000 (A3)

DRAWING No G00114-01

REVISION A

SHEET 1 of 1