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Fraser Thomas
ENGINEERS • RESOURCE MANAGERS • SURVEYORS

SUBMISSION ON THE PROPOSED
SELWYN DISTRICT PLAN,
EAST MADDISONS ROAD,
ROLLESTON

GEOTECHNICAL INVESTIGATION REPORT

Dunweavin 2020 Ltd

SUBMISSION ON THE PROPOSED SELWYN DISTRICT PLAN, EAST MADDISONS ROAD, ROLLESTON

GEOTECHNICAL INVESTIGATION REPORT

Project No.	CH00676	Approved for Issue	
Version No.	1	Name	M V Reed
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SUMMARY

This report presents the results of a geotechnical investigation and appraisal undertaken for the proposed rezoning of the site at East Maddisons Road, Rolleston. The subject site (approximately 13 ha) consists of the following existing properties:

1. Lot 1 DP 26880 (605 East Maddisons Road); approximately 4.86 ha,

2. Lot 2 DP 74311 (617 East Maddisons Road); approximately 4.067 ha,

3. Lot 3 DP 74311 (627 East Maddisons Road); approximately 4.047 ha.

It is understood that it is proposed to lodge a submission on the Proposed Selwyn District Plan, seeking rezoning of the above property from “General Rural” to “General Residential”, to enable future subdivision of the site to create new lots, with an average lot size of approximately 650 m^2, and some medium density lots with a lot size ranging between 400 m^2 and 499 m^2.

The approximate location and extent of the subject site is shown on the appended Fraser Thomas Ltd drawing G00676-01.

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

Foundation design 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 residential development at the subject site.

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

SUBMISSION ON THE PROPOSED SELWYN DISTRICT PLAN
EAST MADDISONS ROAD,
ROLLESTON

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation and appraisal undertaken for the proposed rezoning of the site at East Maddisons Road, Rolleston. The subject site (approximately 13 ha) consists of the following existing properties:

1. Lot 1 DP 26880 (605 East Maddisons Road); approximately 4.86 ha,

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It is understood that it is proposed to lodge a submission on the Proposed Selwyn District Plan, seeking rezoning of the above property from “General Rural” to “General Residential”, to enable future subdivision of the site to create new lots, with an average lot size of approximately 650 m^2, and some medium density lots with a lot size ranging between 400 m^2 and 499 m^2.

The subject site is located on the south-western side of East Maddisons Road.

The approximate location and extent of the subject site is shown on the appended Fraser Thomas Ltd drawing G00676-01.

The subsurface conditions underlying the subject site have been investigated by means of eight hand augered boreholes, and associated Dynamic Cone Penetrometer (DCP) scala tests.

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

The purpose of the geotechnical investigation reported herein was to determine the subsoil conditions beneath the subject site as they may affect future residential development, with particular regard to foundation design considerations, and to determine the suitability of the subject site for the residential development, in support of a submission to generally rezone the area from “General Rural” to “General Residential”.

2.0 PREVIOUS REPORTS

A previous report entitled “Review of liquefaction hazard information in eastern Canterbury, including Christchurch City and parts of Selwyn, Waimakariri and Hurunui Districts”, dated December 2012, was prepared by the Institute of Geological and Nuclear Sciences Limited (GNS Science) for the Environment Canterbury Regional Council.

The December 2012 report was prepared in order to determine the parts of the Canterbury area which may be susceptible to the damaging effects of liquefaction induced ground deformations and areas where liquefaction induced damage is unlikely to occur.

Figure 2.1 presented in the December 2012 report, indicates that the subject site is sited in the zone where the December 2012 report indicates that damaging liquefaction induced ground deformation is considered to be "unlikely". The December 2012 report goes on to state the following with regard to the zone which the subject site is located in close proximity to:

> *"… in this area there is little or no likelihood of damaging liquefaction occurring during strong ground shaking. This assessment area consists of the western part of the project area, and most of Banks Peninsula. Within this area, investigations in most cases can be designed primarily for other geotechnical hazards. Liquefaction however must at least be considered by the geotechnical professional in all cases."*

3.0 SUMMARY OF 2010/2011 DAMAGING CANTERBURY EARTHQUAKE EVENTS

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

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

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

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

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

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

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

4.0 GEOLOGY

In 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 (Q2a)", of Pleistocene age.

The results of the borehole investigations reported herein, in general, indicate that the surficial soils underlying the site are likely to comprise alluvial sediments inferred to be of Pleistocene age.

5.0 PROPOSED SUBDIVISIONAL DEVELOPMENT

As discussed in Section 1.0 of this report, the subject site (approximately 13 ha) consists of the following existing properties:

1. Lot 1 DP 26880 (605 East Maddisons Road); approximately 4.86 ha,
2. Lot 2 DP 74311 (617 East Maddisons Road); approximately 4.067 ha,
3. Lot 3 DP 74311 (627 East Maddisons Road); approximately 4.047 ha.

It is understood that it is proposed to lodge a submission on the Proposed Selwyn District Plan, seeking rezoning of the above property from “General Rural” to “General Residential”, to enable future subdivision of the site to create new lots, with an average lot size of approximately 650 m^2, and some medium density lots with a lot size ranging between 400 m^2 and 499 m^2.

The approximate location and extent of the subject site is shown on the appended Fraser Thomas Ltd drawing G00676-01.

6.0 FIELD INVESTIGATION

6.1 GENERAL

The field investigation comprised a visual appraisal, eight hand augered boreholes, and associated Dynamic Cone Penetrometer (DCP) scala tests.

The approximate locations of the investigation test positions are shown on Fraser Thomas Ltd drawing G00676-01.

6.2 RESULTS OF VISUAL APPRAISAL

A visual appraisal of the subject site was undertaken by a Fraser Thomas Ltd engineering geologist on 3 December 2020.

The subject site is located on the south-western side of East Maddisons Road.

The topography within the subject site is generally flat. The subject site is generally vegetated with grass and hedgerows of mature trees.

An existing shallow water race extends through the site. The water race banks are generally subvertical and approximately 0.3 m in vertical height. The water race is up to approximately 2.5 m wide. The water race banks are unretained. No obvious signs of any significant instability of the water race banks was observed at the time of the investigation reported herein.

The approximate inferred location and extent of the water race, as it affects the subject site, is shown on the appended drawing G00676-01.

The approximate inferred location and extent of the water race, as it affects the subject site, is shown on the appended drawing G00676-01.

Several existing dwellings and associated detached structures are located at the site.

The approximate inferred locations and extents of the existing dwellings, structures and other site features are shown on drawing G00676-01.

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

6.3 HAND AUGERED BOREHOLE INVESTIGATION

Eight hand augered boreholes, numbered H1 to H8 inclusive, were put down at the site, in order to determine the nature and consistency of the subsoils underlying the site.

The boreholes were put down by a qualified Fraser Thomas Ltd engineering geologist. The logs of the boreholes are presented in Appendix A of this report.

The boreholes were generally terminated, when the soils became too difficult to auger, at depths ranging between approximately 0.2 m and 0.6 m below the ground surface existing at the time of the investigation reported herein (i.e. the existing ground surface).

All soils in the boreholes were carefully logged.

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

Dynamic Cone Penetrometer (DCP) scala tests were undertaken from the surface adjacent to the boreholes.

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

The approximate locations of Boreholes H1 to H8 inclusive are shown on drawing G00676-01.

7.0 SUBSURFACE CONDITIONS

7.1 GENERAL

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

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

7.2 TOPSOIL

A surficial layer of topsoil, generally comprising sandy silts, was encountered to a depth of between approximately 0.2 m and 0.3 m below the existing ground surface, at the locations of the boreholes.

7.3 ALLUVIAL SEDIMENTS

The results of the field investigations reported herein indicate that the surficial topsoil is generally underlain by material, inferred to comprise sandy gravels. These soils were generally encountered at depths ranging between approximately 0.2 m and 0.6 m below the existing ground surface, at the locations of the boreholes. The hand augered boreholes were not able to be progressed through these soils.

The results of the DCP tests undertaken in the sandy gravels, at the locations of the boreholes, generally obtained DCP blow counts of between approximately 7 and greater than 30 blows per 50 mm penetration, corresponding to a SPT 'N' value of generally greater than 50, corresponding to a very dense consistency.

The logs of existing water bore logs, put down in the vicinity of the subject site, have also been sourced from Environment Canterbury records. The existing water bore logs indicate that sandy gravels are generally located at shallow depths, which is consistent with the subsoil conditions encountered at the subject site. The bore logs indicate that these sandy gravels generally extend to depths in excess of approximately 36 m below the ground surface. Based on the foregoing, it is, in our opinion, likely that the gravel soils underlying the site extend to significant depths below the existing ground surface.

A thin layer of cohesive soils (100 mm to 300 mm thickness), generally comprising gravelly sandy silts, was also encountered at the locations of some boreholes, below the topsoil layer and on top of the underlaying sandy gravels.

In situ undrained shear strength values in excess of 200 kPa were generally measured in these sediments, using hand held shear vane equipment, corresponding to a hard consistency.

7.4 GROUNDWATER

Groundwater was not encountered at the locations of the hand augered boreholes, during the investigation reported herein. However, based on information obtained from the existing water bore logs in the vicinity of the subject site, the groundwater level is inferred to be at a depth in excess of approximately 10 m below the existing ground surface.

8.0 LIQUEFACTION POTENTIAL ASSESSMENT

8.1 GENERAL

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

The two main effects of liquefaction on soils are:

(a) Consolidation of the liquefied soils,

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

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

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

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

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

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

8.2 METHOD OF ANALYSIS

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

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

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

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

(i) Step 1: Assessment of liquefaction susceptibility,

(ii) Step 2: Triggering of liquefaction,

(iii) Step 3: Consequences of liquefaction.

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

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

8.3 ASSESSMENT OF LIQUEFACTION SUSCEPTIBILITY

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

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

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

(c) Areas with a high groundwater level.

As discussed in Section 4.0 of this report, the geological map for the Christchurch area indicates that the site is likely to be underlain by “brownish-grey river alluvium (Q2a)”, of Pleistocene age.

As discussed in Section 7.3 of this report, the results of the hand augered borehole investigations indicate the site is generally underlain by very dense sandy gravels.
As discussed in Section 7.4 of this report, the groundwater level is inferred to be at a depth in excess of approximately 10 m below the existing ground surface.

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

9.0 FOUNDATION DESIGN CONSIDERATIONS

9.1 GENERAL

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

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

Notwithstanding, it is anticipated that a site specific geotechnical investigation will be required to be undertaken, for any new building proposed to be constructed at the subject site, in support of an application for building consent.

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

As discussed in Section 8.3 of this report, it is our opinion that the 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 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 (as modified by B1/AS1), and in accordance with the recommendations presented in this report.

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

9.3 SHALLOW FOUNDATIONS LOCATED IN CLOSE PROXIMITY TO THE EXISTING WATER RACE AT THE SITE

As discussed in Section 6.2 of this report, an existing shallow water race extends through the site. The water race banks are generally subvertical and approximately 0.3 m in vertical height. The water race is up to approximately 2.5 m wide. The approximate inferred location and extent of the water race, as it affects the subject site, is shown on the appended drawing G00676-01.

Recent alluvial sediments are likely to have been deposited in the base of the water race, and also possibly in the immediate vicinity of the water race.

Due to the likely variable nature of recent alluvial sediments and the likely presence of highly compressible sediments, there is, in our opinion, a risk that shallow building foundations founded on recent alluvial sediments 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 recent alluvial sediments, 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 crest of any water race side slopes at the site.

It should be noted, should the site be subject to residential development, that the subdivisional earthworks would likely involve the stripping of the water race and the backfilling of the water race with engineered fill material. Providing the earthworks are undertaken appropriately, the backfilling of the water race would result in the removal of the requirement for any horizontal offset from the ware race, for shallow foundation design purposes.

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. It is recommended that a strength reduction factor (Φ_{bc}) of 0.5 be adopted for limit state design in accordance with the requirements of AS/NZS 1170, resulting in a design (dependable) bearing capacity value of 150 kPa.

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

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

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

11.0 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.

12.0 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.

13.0 STORMWATER AND EFFLUENT DISPOSAL

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

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 residential development at the subject site.

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

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 subdivision.

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

(c) The results of the field investigations reported herein indicate that the surficial topsoil is generally underlain by material, inferred to comprise very dense sandy gravels. These soils were generally encountered at depths ranging between approximately 0.2 m and 0.6 m below the existing ground surface, at the locations of the boreholes. The hand augered boreholes were not able to be progressed through these soils.

The logs of existing water bore logs, put down in the vicinity of the subject site, have also been sourced from Environment Canterbury records. The existing water bore logs indicate that sandy gravels are generally located at shallow depths, which is consistent with the subsoil conditions encountered at the subject site. The bore logs indicate that these sandy gravels generally extend to depths in excess of approximately 36 m below the ground surface. Based on the foregoing, it is, in our opinion, likely that the gravel soils underlying the site extend to significant depths below the existing ground surface.

(d) Groundwater was not encountered at the locations of the hand augered boreholes, during the investigation reported herein. However, based on information obtained from the existing water bore logs in the vicinity of the subject site, the groundwater level is inferred to be at a depth in excess of approximately 10 m below the existing ground surface.

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

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

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

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

(b) In order to mitigate the risk of any proposed future shallow foundations being adversely affected by the settlement of recent alluvial sediments, 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 crest of any water race side slopes at the site.

It should be noted, should the site be subject to residential development, that the subdivisional earthworks would likely involve the stripping of the water race and the backfilling of the water race with engineered fill material. Providing the earthworks are undertaken appropriately, the backfilling of the water race would result in the removal of the requirement for any horizontal offset from the ware race, for shallow foundation design purposes.

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

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.

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

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

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

(f) It is recommended that any temporary excavated slopes be constructed to a maximum batter slope of 45^{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.

15.0 LIMITATIONS

The professional opinion expressed herein has been prepared solely for, and is furnished to our client, Dunweavin 2020 Ltd, 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

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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

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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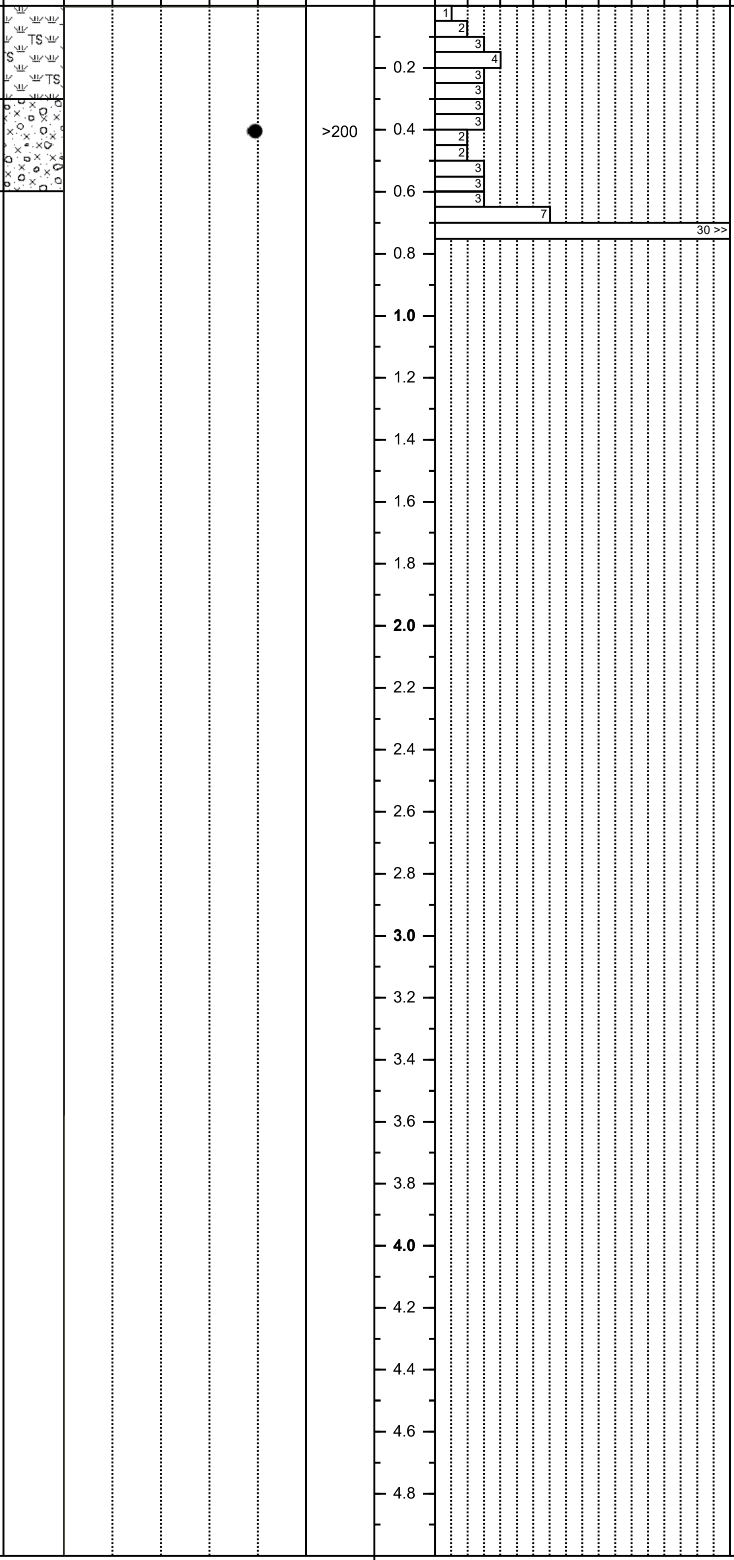
HAND AUGER LOG

Hole No: H1

Project No:	**Project:**	**Shear Vane:**	**Date Drilled:**	**Logged By:**	**Checked By:**
CH00676	Dunweavin 2020 Ltd East Maddisons Road, Rolleston	2512	03/12/2020	SG	

Depth (m)	Description of Strata	Geological Unit	Groundwater
0.0 – 0.3	SILT, sandy (fine), brown, dry, rootlets [TOPSOIL]	T/S	GWNE
0.3 – 0.6	SILT, sandy (fine to medium), gravelly (fine to medium), brown, hard, dry [ALLUVIAL SEDIMENTS]	River Alluvium	
0.6	EOH: 0.60 m TDTA - GRAVELS ENCOUNTERED		



Remarks:
Groundwater not encountered on 03/12/2020.

Datum:

Coordinates:

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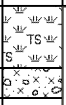
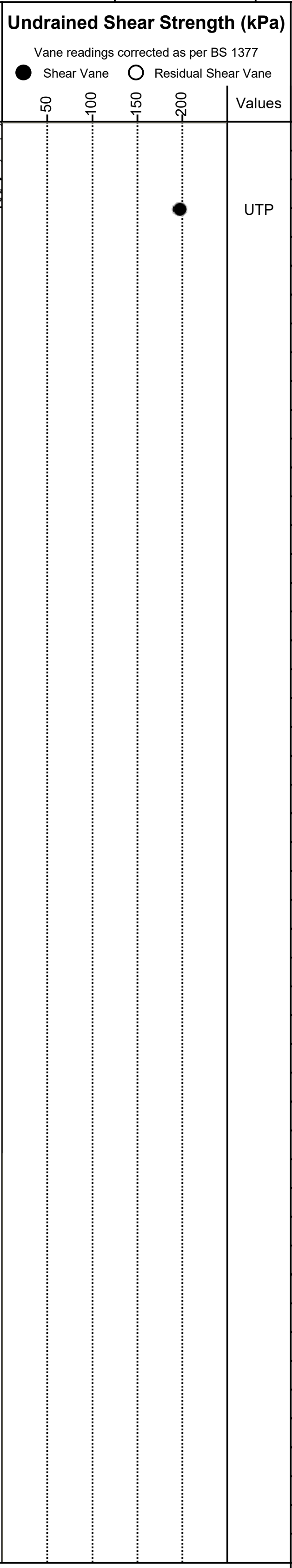
HAND AUGER LOG

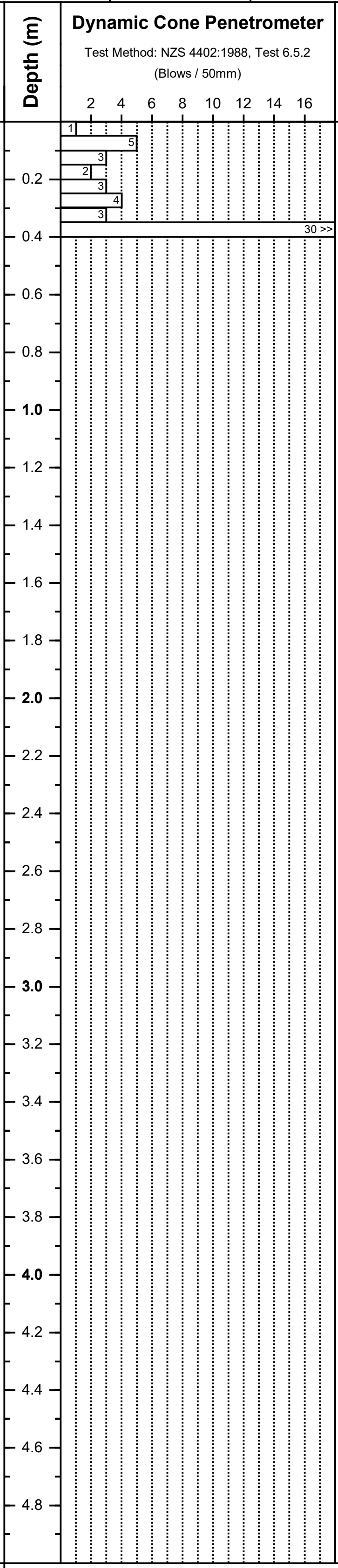
Hole No: **H2**

Project No:	**Project:**	**Shear Vane:**	**Date Drilled:**	**Logged By:**	**Checked By:**
CH00676	**Dunweavin 2020 Ltd** **East Maddisons Road, Rolleston**	2512	03/12/2020	SG	

Description of Strata	**Geological Unit**	**Groundwater**
SILT, sandy (fine), brown, dry, rootlets [TOPSOIL]	T/S	GWNE
SILT, sandy (fine to medium), gravelly (fine to medium), brown, hard, dry [ALLUVIAL SEDIMENTS]		
EOH: 0.30 m TDTA - GRAVELS ENCOUNTERED		







Remarks:
Groundwater not encountered on 03/12/2020.

Datum:

Coordinates:

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HAND AUGER LOG

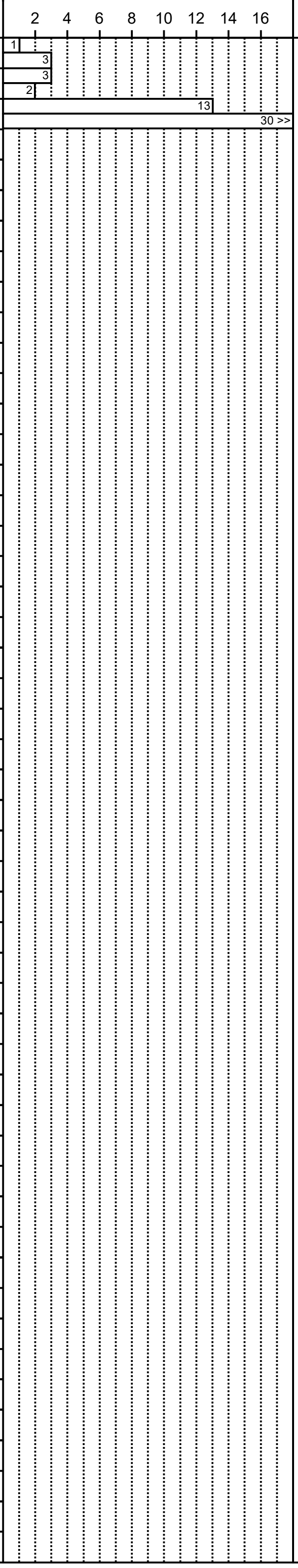
Hole No: **H3**

Project No:	**Project:**	**Shear Vane:**	**Date Drilled:**	**Logged By:**	**Checked By:**
CH00676	Dunweavin 2020 Ltd East Maddisons Road, Rolleston		03/12/2020	SG	

Depth (m)	Description of Strata	Geological Unit	Graphic Log	Undrained Shear Strength (kPa)	Groundwater
0.0 – 0.2	SILT, sandy (fine), brown, dry, rootlets [TOPSOIL]	T/S	TS		GWNE
0.2	EOH: 0.20 m TDTA - GRAVELS ENCOUNTERED				

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:
Groundwater not encountered on 03/12/2020.

Datum:

Coordinates:

Generated with CORE-GS by Geroc - Hand Auger MASTER - 4/12/2020 9:52:15 AM

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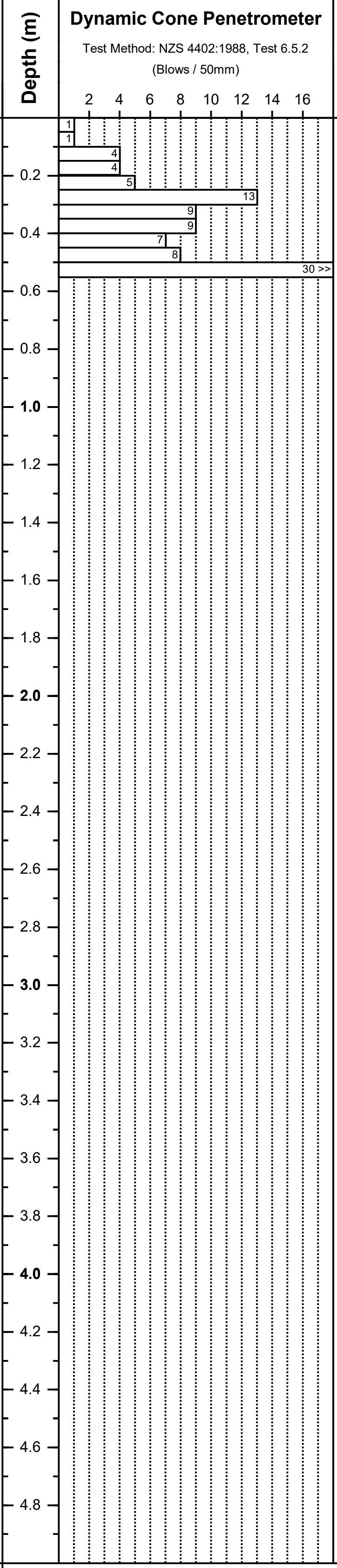
HAND AUGER LOG

Hole No:
H4

Project No:	**Project:**	**Shear Vane:**	**Date Drilled:**	**Logged By:**	**Checked By:**
CH00676	**Dunweavin 2020 Ltd** **East Maddisons Road, Rolleston**		03/12/2020	SG	

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	**Groundwater**
	SILT, sandy (fine), brown, dry, rootlets [TOPSOIL]	T/S	TS			GWNE
0.2	EOH: 0.20 m TDTA - GRAVELS ENCOUNTERED					



Remarks:
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Datum:

Coordinates:

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HAND AUGER LOG

Hole No: H5

Project No: CH00676	**Project:** Dunweavin 2020 Ltd East Maddisons Road, Rolleston	**Shear Vane:**	**Date Drilled:** 03/12/2020	**Logged By:** SG	**Checked By:**

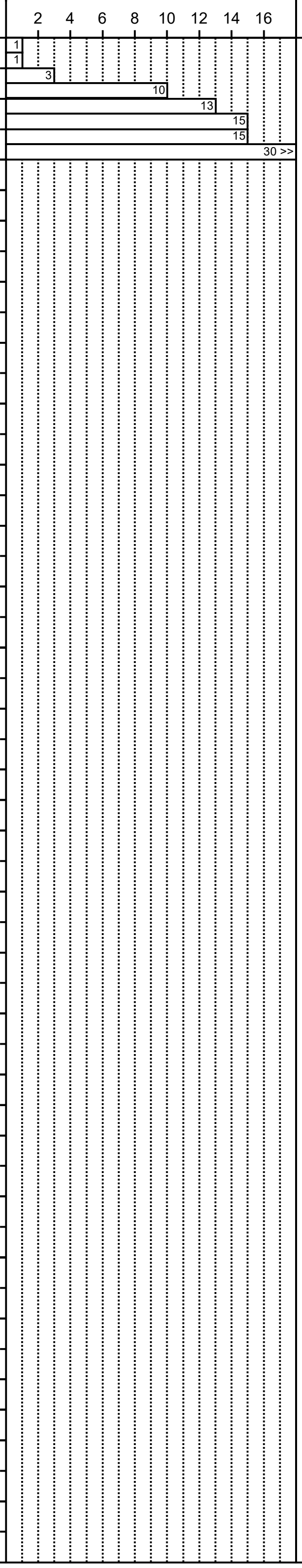
Depth (m)	Description of Strata	Geological Unit	Graphic Log	Undrained Shear Strength (kPa)	Values	Groundwater
0.0 – 0.2	SILT, sandy (fine), brown, dry, rootlets [TOPSOIL]	T/S	TS			GWNE
0.2	EOH: 0.20 m TDTA - GRAVELS ENCOUNTERED					

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:
Groundwater not encountered on 03/12/2020.

Datum:

Coordinates:

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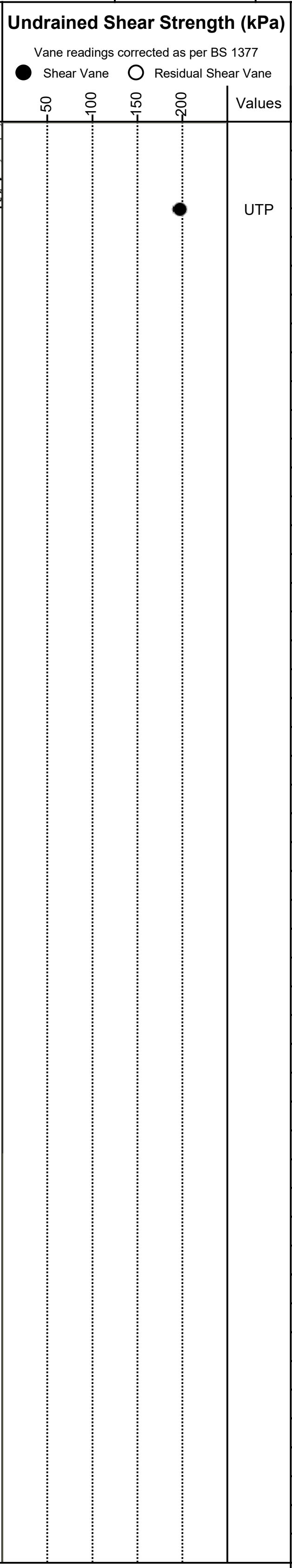
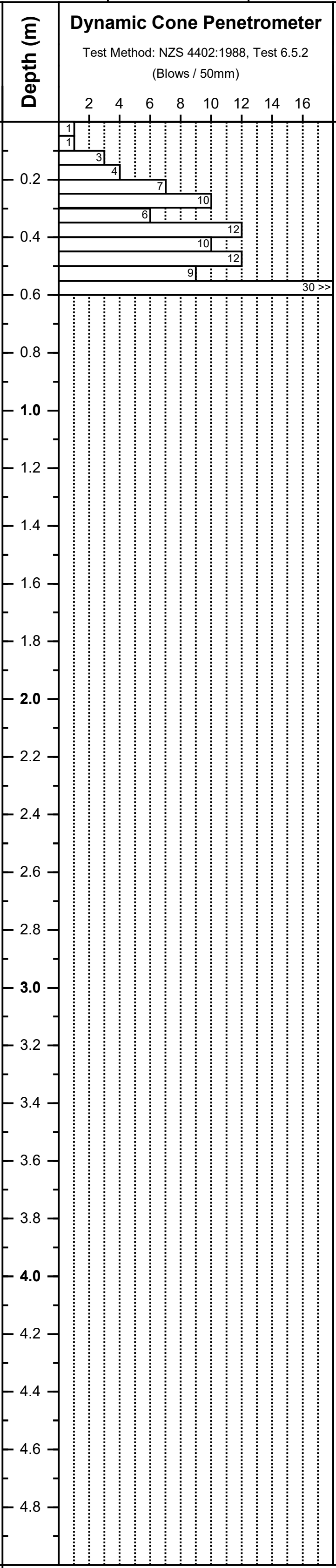
HAND AUGER LOG

Hole No: H6

Project No:	**Project:**	**Shear Vane:**	**Date Drilled:**	**Logged By:**	**Checked By:**
CH00676	**Dunweavin 2020 Ltd** **East Maddisons Road, Rolleston**	2512	03/12/2020	SG	

Description of Strata	Geological Unit	Undrained Shear Strength Values	Groundwater
SILT, sandy (fine), brown, dry, rootlets [TOPSOIL]	T/S		GWNE
SILT, sandy (fine to medium), gravelly (fine to medium), brown, hard, dry [ALLUVIAL SEDIMENTS]		UTP	
EOH: 0.30 m TDTA - GRAVELS ENCOUNTERED			





Remarks:
Groundwater not encountered on 03/12/2020.

Datum:

Coordinates:

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HAND AUGER LOG

Hole No: H7

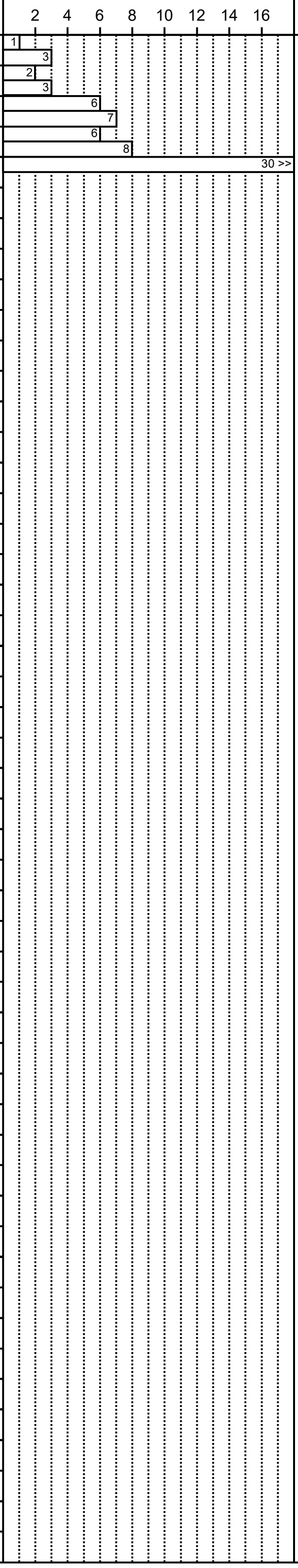
Project No:	**Project:**	**Shear Vane:**	**Date Drilled:**	**Logged By:**	**Checked By:**
CH00676	Dunweavin 2020 Ltd East Maddisons Road, Rolleston		03/12/2020	SG	

Depth (m)	Description of Strata	Geological Unit	Graphic Log	Undrained Shear Strength (kPa) Values	Depth (m)	Groundwater
0.0–0.2	SILT, sandy (fine), brown, dry, rootlets [TOPSOIL]	T/S	TS			GWNE
0.2	EOH: 0.20 m TDTA - GRAVELS ENCOUNTERED					

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

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

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



Remarks:
Groundwater not encountered on 03/12/2020.

Datum:

Coordinates:

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HAND AUGER LOG

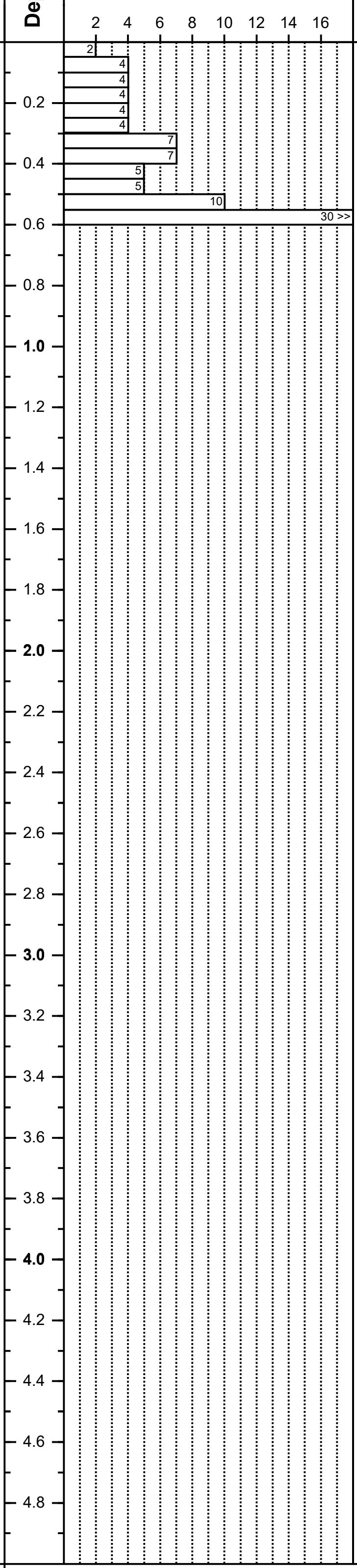
Hole No: H8

Project No: CH00676	**Project:** Dunweavin 2020 Ltd East Maddisons Road, Rolleston	**Shear Vane:**	**Date Drilled:** 03/12/2020	**Logged By:** SG	**Checked By:**

Depth (m)	Description of Strata	Geological Unit
0.0–0.2	SILT, sandy (fine), brown, dry, rootlets [TOPSOIL]	T/S
0.2–0.3	SILT, sandy (fine to medium), gravelly (fine to medium), brown, hard, dry [ALLUVIAL SEDIMENTS]	
	EOH: 0.30 m TDTA - GRAVELS ENCOUNTERED	

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)

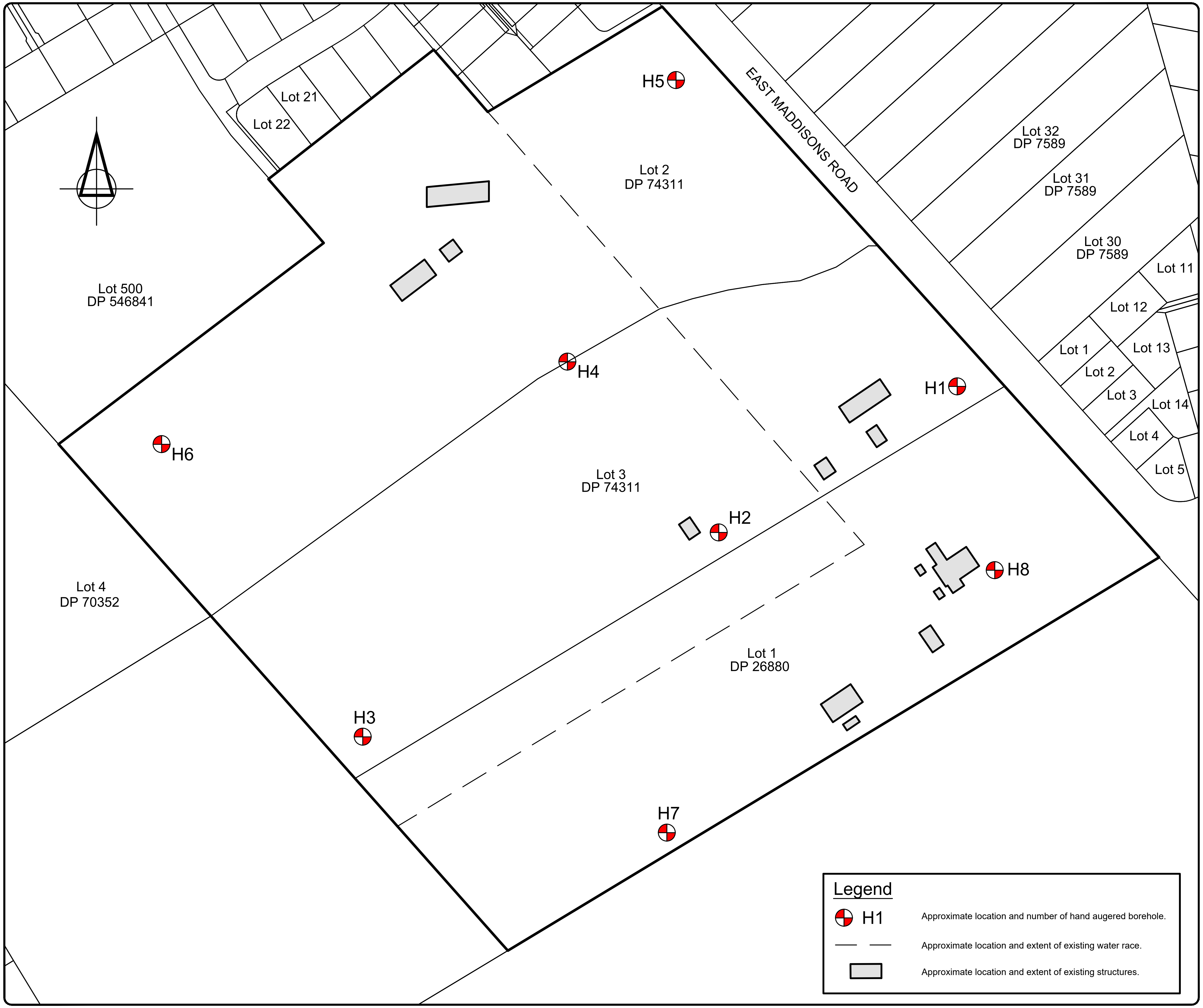


Groundwater: GWNE

Remarks:
Groundwater not encountered on 03/12/2020.

Datum:

Coordinates:



Legend

⊕ H1	Approximate location and number of hand augered borehole.
— —	Approximate location and extent of existing water race.
▭	Approximate location and extent of existing structures.

SURVEYED		
DESIGNED		
DRAWN	KT	02/12/2020
CAD	GS	07/12/2020
CHECKED		

APPROVED DATE

MVR

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

DUNWEAVIN 2020 LTD

PROJECT

EAST MADDISONS ROAD
ROLLESTON

TITLE

SITE PLAN

Fraser
Thomas

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SCALE 1:1750 (A3)

DRAWING No G00676-01

REVISION A

SHEET 1 of 1

Dunweavin 2020 Ltd

Fraser Thomas
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SUBMISSION ON THE PROPOSED
SELWYN DISTRICT PLAN,
EAST MADDISONS ROAD,
ROLLESTON

PRELIMINARY SITE INVESTIGATION – CONTAMINATION

Dunweavin 2020 Ltd

SUBMISSION ON THE PROPOSED
SELWYN DISTRICT PLAN,
EAST MADDISONS ROAD,
ROLLESTON

PRELIMINARY SITE INVESTIGATION - CONTAMINATION

Project No.	CH00676	Approved for Issue	
Version No.	1	Name	Sean Finnigan
Status	Final	Signature	
Authors	S Gladwin		
Reviewer	S Finnigan	Date	10 December 2020

Fraser Thomas Limited
Consulting Engineers, Licensed Surveyors
Planners & Resource Managers
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PO Box 204006, Highbrook, Auckland, 2025
Auckland, New Zealand
Tel : +64 9 278-7078 : Fax : +64 9 278-3697
Email: sfinnigan@ftl.co.nz

DUNWEAVIN 2020 LTD

SUBMISSION ON THE PROPOSED SELWYN DISTRICT PLAN EAST MADDISONS ROAD, ROLLESTON

PRELIMINARY SITE INVESTIGATION - CONTAMINATION

EXECUTIVE SUMMARY

In response to instructions from Dunweavin 2020 Ltd, Fraser Thomas Limited (FTL) undertook a Preliminary Site Investigation (PSI) for Lot 1 DP 26880, Lot 2 DP 74311, Lot 3 DP 74311 ('site'). The site comprises three properties and is located on the south-western side of East Maddisons Road.

This investigation involved a desktop study, site walkover, and reporting associated with potential land contamination issues.

The main rationale and objectives for this investigation were:

- To identify the main actual or potential contamination issues due to ongoing and historic use of land within the site.
- To confirm that the site is suitable or can be made suitable for the proposed rezoning.

This investigation has been managed, reviewed and approved by a Suitably Qualified and Experienced Practitioner (SQEP), as defined in the National Environmental Standard for Assessing and Managing Contaminants in Soil to Protect Human Health (NESCS).

The NESCS governs a number of activities, including soil sampling, soil disturbance, subdivision and changes of land use on potentially contaminated land in New Zealand. In general, the rules of the NESCS apply to sites on which it is "more likely than not" that a HAIL (Hazardous Activities and Industries List) activity has occurred or is occurring (Regulation 5(7).

This investigation has confirmed that the majority of the subject site has only been used for grazing purposes. The NESCS does not apply to these portions of the site under Regulation 5(7).

This investigation has however, identified a few localised potential or actual HAIL activities:

- *Activity A17: Storage tanks or drums for fuel, chemicals or liquid waste.* This relates to the inferred fuel source for the boiler believed to have historically been located in Lot 1 26880.
- *Activity F4: Motor vehicle workshops* This relates to the vehicle workshop and empty oil containers located in Lot 2 DP 74311.
- *Activity I: Land subject to intentional or accidental release of hazardous substances in sufficient quantity that it could be a risk to human health or the environment:* This

relates to the deteriorated condition of the paint on the older existing buildings on Lot 1 DP 26880. Additionally, other activities such as burn piles may have resulted in release of hazardous substances.

- *Activity E1: Asbestos product manufacture or disposal including sites with buildings containing asbestos products know to be in a deteriorated condition.* This relates to the demolished/removed building, deteriorated state of existing older buildings and alterations to existing dwelling on Lot 1 DP 26880, due to the fact that these buildings have been present since the early 1970s, and the dwelling appears to have had extension work undertaken at some stage.

In summary, based on the information presented in this report, whilst it is clear that historic HAIL activities have occurred at the site, it is uncertain what effects, if any, they have had on site soils. Therefore, in accordance with NESCS requirements, a Detailed Site Investigation (DSI) is required in order to assess site soils to determine environmental effects, or otherwise. This should be undertaken as part of a future subdivision consent application. If further investigation is not undertaken prior to lodging for resource consent, then any future subdivision would be a discretionary activity under Regulation 11 of the NESCS.

Copyright of this report is held by Fraser Thomas Ltd. The professional opinion expressed herein has been prepared solely for, and is furnished to our client and Environment Canterbury (this being a regional planning requirement), on the express condition that it will only be used for the works and the purpose for which it is intended.

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