# Geotechnical and Earthworks Assessment for Proposed Residential Subdivision at Parkburn Quarry, 922 Luggate-Cromwell Road (SH6), Cromwell

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|---|---|
| Prepared for: | Fulton Hogan Land Development Ltd |
| Date: | 16/07/2026 |
| Our Reference Number: | 24019_3 V4 |
| Report prepared by: | [redacted] CMEngNZ, CPEng |
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# Author Credentials

## [redacted] Statement of Qualifications and Experience

I am a Director of JKCM Ltd t/a Insight Engineering.

I hold a Bachelor of Engineering from the University of Stellenbosch, completed in 2004. I am a Chartered Professional Engineer and a chartered member of Engineering New Zealand and have held this status since 2012.

I have 19 years professional experience in Geotechnical Engineering for land development projects, including preliminary and detailed geotechnical investigations, geotechnical numerical analyses for slopes, deep excavations and liquefiable land, design of retaining structures and ground improvement systems as well as earthworks monitoring and certification.

I have acted as lead Geotechnical Engineer for a wide variety of land development projects, ranging from single sites to large, multi-lot residential, commercial and industrial properties in Auckland, Canterbury, Southland and Otago.

In my role as author of this report, I confirm that I have read and comply with the Environment Court of New Zealand's Code of Conduct for Expert Witnesses Practice Note 2023.

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# 1 Introduction and Scope of Report

JKCM Ltd, trading as Insight Engineering (IE), was requested to undertake a geotechnical assessment 922 Luggate-Cromwell Road, Cromwell (herein referred to as 'the site').

We understand that the site is proposed to be subdivided and developed for residential use, as per the development and scheme plans included in the Fast Track substantive application (refer also to Section 3 of this report). Therefore, IE was requested to undertake a geotechnical assessment of the site as well as the proposed earthworks that will be required to facilitate the development of the site.

Our scope of work for this geotechnical assessment included:

- Review and summarise publicly available information for the site, including published geological maps, local authority GIS webmaps and records, the existing geotechnical reports for the site, the NZGD and historical aerial photographs, as well as our own geotechnical records;
- Assessment of general geotechnical considerations for the proposed residential development at the site, including mapped or observed hazards and bulk earthworks considerations;
- Observe field compaction trials to inform the earthworks methodologies and specifications, and;
- Preparation of this geotechnical report, presenting the findings of our review and recommendations for the proposed subdivision to accompany a Resource Consent application.

Assessment of the Park Burn flooding potential and likely effects on the proposed development is outside of the scope of this report, and it is our understanding that this is being assessed by others (Refer to Sections 4.2.5 and 7.2 below).

This report is not intended to express an opinion on the suitability or otherwise of any future lots or to provide detailed geotechnical information, for example, as might be required to accompany a Building Consent application. It is IE's expectation that a series of Geotechnical Completion Reports will be prepared following completion of each stage of earthworks, and these reports will present lot specific geotechnical information for the design of future structures, and identify any areas requiring further geotechnical assessment.

# 2 Site Description

The site is located at 922 Luggate-Cromwell Road (SH6), Cromwell (legal description PT SEC 63 BLK IV WAKEFIELD SD). Figure 1 below shows the site outlined in red.

The site is located on the western side of Lake Dunstan/Te Wairere and is approximately 8 km north of Cromwell. It is bound by rural resource land to the north and west (beyond SH6). The Park Burn creek flows through the site in a roughly NW to SE direction and it discharges into Lake Dunstan/Te Wairere.

Most of the site is zoned as low and medium density residential following Plan Change 21 in 2024. The northern strip is zoned as industrial; however we understand that another Private Plan Change application is currently in progress to re-zone this portion of the site to residential. Approximately 4.2 ha near the eastern side is zoned as Business Area 1-2.

The Parkburn Quarry, which was established in the 1980s and is still operational at the time of writing, currently occupies about half of the site area. Also, present are offices and yards for the quarry operators (Fulton Hogan Ltd), a concrete plant, and an asphalt plant. The area to the east of the Park Burn is used as a “cleanfill” disposal area (for imported materials), and general materials and machinery storage.

Part of the site to the north of the quarry area and south of the offices is generally undisturbed and is considered as “greenfield” land. Areas that were previously quarried have been backfilled with pea gravel in places which is generally a by-product of the quarry operations. These areas are typically either flat or gently sloping, with the surface stabilised with topsoil and vegetation.

The north-eastern end of the site is designated for “cleanfill” disposal and general storage. This area, as well as the land containing the concrete and asphalt plants, are verified on the HAIL register.

It should be noted that some areas within the operational quarry area shown on Figure 2 have been previously quarried and backfilled with pea gravel, however currently contain ponds or are otherwise used by the quarry operator. The “cleanfill” area at the north-eastern end was also previously quarried and backfilled with pea gravel to varying extents, with uncontrolled imported fill overlying the pea gravel.

There are bunds located at the western side of the site adjacent to SH6 that act as a visual barrier. These are understood to be formed from mixtures of topsoil and silty overburden soils scraped from the quarry. Similar bunds are located at the eastern sides of the site.

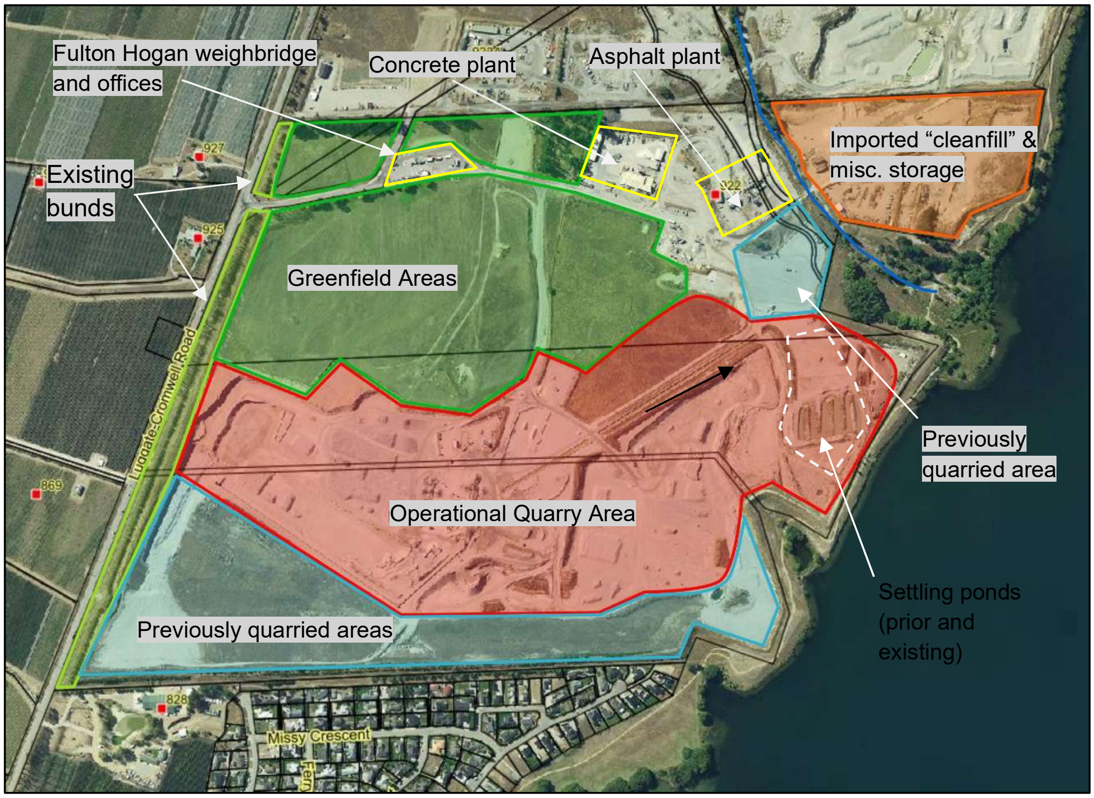
The existing (undisturbed) surface elevations are typically around 205 mRL, with the cleanfill area being around 200 mRL. The base of the quarry is generally around 195 mRL or 1 m above the lake level.

The site location and general site layout are presented as Figures 1 and 2 below.

## Figure 1: Site Location Plan (not to scale)


Images sourced from Central Otago District Council GIS and Google

Figure 2: Simplified Site Layout (at time of writing)




Base image sourced from Central Otago District Council GIS

## 3 Development Proposals

The proposed development includes bulk cut to fill earthworks to form ground surfaces to support future residential and commercial properties as indicated by the plans prepared by LandPro Ltd. Associated roading, green spaces and reserves are also proposed, including a 50 m wide exclusion zone proposed at the western boundary adjacent to SH6. Two coves are proposed, with the northern cove being up to 4 m deep and the southern cove being no more than 1.5 m deep.

The earthworks plans indicate cut to fill activities to maximum depths of 15 m. The earthworks are proposed to be completed in four stages, commencing at the southern end of the site and gradually progressing northwards.

Stage 1 encompasses previously quarried areas and part the existing operational area. Stage 2 includes part of the greenfield area and part of the operational area. Stage 3 includes the portion of the site along the northern boundary on the western side of the Park Burn, including the existing offices, and concrete and asphalt plants. Stage 4 includes the “cleanfill” disposal area on the eastern side of the Park Burn.

Remediation areas RA1-RA3 are shown on the earthworks plans. This refers to previously quarried areas that are known to have been backfilled with pea gravel, which is not a suitable engineered fill material. Geotechnical remediation is likely to involve over-excavation and sorting of the existing materials, removal of unsuitable materials, crushing of pea gravel, and replacement of uncontrolled fill with engineered, crushed materials. Further discussion on this process is presented in Sections 6 and 7 below.

The fill depths indicated on the earthworks plans have been estimated based on an assumed worst-case scenarios for the various remediation areas where the assumed depth of prior quarry activities was to ~ 195 mRL.

The Stage 4 area also contains existing pea gravel backfill, as well as other imported fill materials. These materials have been placed in an uncontrolled manner, and therefore, geotechnical remediation of this area is also required. Due to the HAIL status of this portion of the site, it is proposed that these works be carried out in conjunction with a Suitably Qualified Environmental Practitioner (SQEP).

The finished site gradients are indicated to be gently sloping (<10%). Some steeper slopes up to 50% are proposed around the lakeside edges of the development. We understand that these slopes are likely to be retained, or depending on the ground conditions, height of the slopes and distance to buildings/infrastructure, they may be regraded, reinforced with geogrids or left unsupported.

The proposed coves are likely to be formed by excavation and filling to form the batters adjacent to the waterways, once the temporary stormwater and sediment works are completed. Excavation below ~195 mRL is required to achieve the design depth below the water line for both coves. This is likely to involve combination of excavation, pumping and dredging, until the design configuration is achieved. Breaking through to the lake will be the final step in the formation of the coves.

# 4 Desktop Review

## 4.1 Geology

### 4.1.1 Geological Setting

The Otago region is predominantly underlain by metamorphic basement rocks that have been folded and uplifted by continental plate boundary tectonic processes. Glacial and fluvial processes have shaped landforms in much of the western Central Otago area where the site is located.

During the Miocene period, Central Otago was a large river and lake environment resulting in deposited sediment which were carried from mountain ranges to the north. The Miocene terrestrial sediments and basement rock are almost exclusively covered by Quaternary sediments (Pleistocene glacial till and outwash deposits) as well as more recent (Holocene) alluvium and fan deposits in the Cromwell Basin.

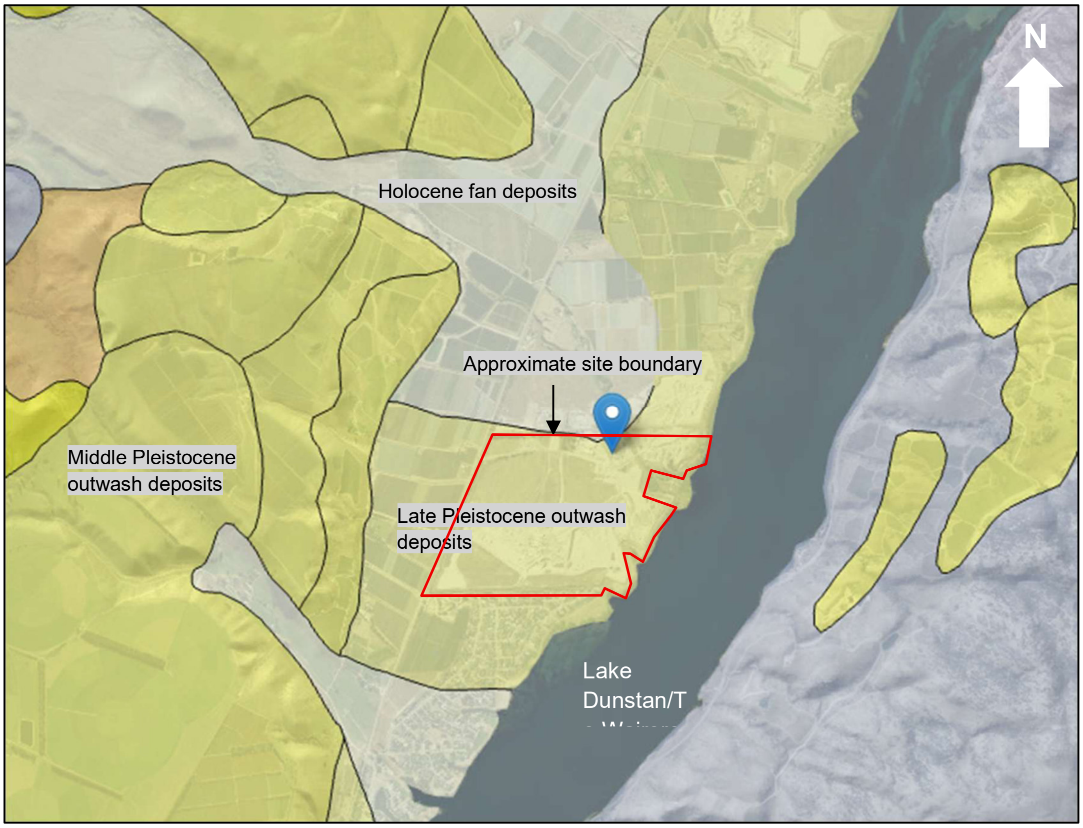
### 4.1.2 Mapped Geology

The majority of the site is mapped[1,2] as being underlain by Late Pleistocene outwash deposits of Albert Town Advance, described as "*Unweathered to slightly weathered, loose, sandy to silty, well rounded gravel usually on large outwash plains*".

Holocene Fan deposits, broadly described as "*Loose, commonly angular, boulders, gravel, sand, and silt forming alluvial fans; grades into scree (upslope) and valley alluvium*", are mapped extending over a small portion of the site at the northern boundary.

The published mapping is reproduced as Figure 3 below.

**Figure 3: Published Geological Mapping**




Base image sourced from GNS Science

## 4.2 Regional and Local Council Reports and GIS Maps

Mapped hazards from the Otago Regional Council (ORC) natural hazards portal are discussed in the following sections.

### 4.2.1 Active Faults

The characteristics and potential effects of local active faults are discussed in two reports[3,4] prepared by GNS for ORC.

The nearest mapped fault zone to the site is the Pisa Fault Zone, located approximately 1.4 km to the west of the site. The Pisa Fault Zone is reported by GNS as having "*definite, likely and possible active fault and fold strands*" with a calculated recurrence interval (RI) of 22,000 years.

Fault Awareness Areas (FAAs) have recently been developed by ORC. The following text is extracted from a 2025 ORC Committee report[5] relating to FAAs: "*The intent of a Fault Awareness Area is that it is sufficiently large to encompass the full range of plausible locations of the active fault, to account for the inaccuracies involved in mapping at 1:250,000 scale, and to accommodate the possibility that future fault ruptures and associated ground deformation may not coincide exactly with mapped areas of previous fault rupture deformation*".

The FAA around the Pisa Fault Zone is 500 m wide, approximately 1.1 to 1.6 km from the western site boundary.

The Dunstan Fault Zone, also mapped as "*definitely*" active is located approximately 18 km to the south-east and has a reported recurrence interval of 8,200 years. FAA around the Dunstan Fault Zone is also 500 m wide.

The GNS report discusses that the faults in the Upper Clutha region are generally assessed as low-activity faults, with many calculated recurrence intervals being more than 10,000 years. However, the faults are capable of generating large, locally damaging earthquakes in the future decades to centuries. Accordingly, severe ground shaking and permanent deformation of the land are possible primary effects in the event of rupture of these faults.

The Alpine fault, located approximately 100 km to the west of the site, also presents a seismic hazard to the Otago region. The Alpine Fault is assessed as having a shorter (more frequent) recurrence interval and capable of generating earthquakes of Mw8.1. Research completed by Howarth *et al*.[6], reports a 75% likelihood for magnitude 7 or greater rupture of the Alpine Fault over the next 50 years.

#### 4.2.2 Seismic Shaking Intensity

The ORC hazards portal provides estimates of ground shaking intensity for various fault rupture scenarios and exceedance for given recurrence intervals. Shaking intensity is presented using the Modified Mercalli Intensity (MMI) scale which is composed of increasing levels of intensity that range from imperceptible shaking to catastrophic destruction. The version of the scale presented on the ORC database goes from MM1 up to MM12.

The source report[7] for the seismic shaking intensity mapping can be found on the ORC website and contains additional detail regarding specific event shaking intensity and the MMI scale.

At the site, MM6 shaking is expected to be exceeded 1/100 years. This level of ground shaking would be felt by all. Effects of MM6 shaking include difficulty walking steadily, objects falling from shelves, slight damage to some types of buildings, trees and bushes shake and loose material may be dislodged from sloping ground.

MM9 shaking is expected to be exceeded 1/2500 years at the site. This level of ground shaking can heavily damage or destroy many buildings, houses not secured to foundations can slide off, ground cracking is conspicuous, and landsliding is possible on steep slopes.

#### 4.2.3 Liquefaction

The site is mapped within a region indicated to have a low to nil risk of liquefaction (A Domain) on the Otago Regional Council (ORC) hazards database. The mapping is based on the report[8] compiled by GNS Science.

#### 4.2.4 Seismic Ground Class

The seismic ground classification appears to be based on the mapped geology and NZS1170.5:2004. The site is mapped by ORC as Class D (deep or soft soil). It should be noted that NZS1170.5 was updated in 2025 and now includes a slightly different site classification system from the prior version.

#### 4.2.5 Alluvial Fans

Regional scale alluvial fan mapping, reproduced as Figure 4 below, extends into the site boundaries. The mapping indicates that an approximately 200 m wide strip of land adjacent to the Park Burn, tapering to practically zero at the creek outlet, is potentially subject to floodwater dominated fan activity. The remainder of the site is excluded from mapped alluvial fan activity, although inactive fan is mapped to the north of the site.

The mapping is based on the Otago Alluvial Fans Project report[9] dated 2009 prepared by Opus Consultants Ltd.

Figure 4: Published Alluvial Fan Mapping


Base image sourced from GNS Science

## 4.3 Groundwater and ORC Well Records

A report[10] prepared by the ORC dated October 2012 discusses that the depth to the water table in the Cromwell basin can vary between 2 m to 34 m, primarily due to variances in the ground surface topography, with the mean depth being approximately 20 m below the ground surface. The report further discusses that the depth to the water table is likely to be shallower closer to the edge of Lake Dunstan/Te Wairere, and that the elevation of the water table "*tends to conform to a surface less than a metre above the mean lake water level of 194.2 m AMSL*".

A review of the Wells Aotearoa New Zealand database[11] has yielded information for several water bores located at and around the site. This information is summarised in Table 1, and the well locations are presented as Figure 5 below.

Table 1: ORC Well Records Summary

| Well Identifier | Driller Name | Well Status | Elevation (datum) | Static Water Level (Elevation) | Well Depth | Date Drilled |
|---|---|---|---|---|---|---|
| G41/0113 | McNeill Drilling | Existing | 202.7 (NZVD 2016) | 16.47 m (186.29) | 24.89 m | 12/08/1992 |
| G41/0125 | McNeill Drilling | Not recorded | 203.154 (Dunedin 1958) | Not recorded | 10 m | 1/01/1990 |
| G41/0115 | Mc Neill Drilling | Existing | 210.5 (Dunedin 1958) | 12.6 m (197.9) | 23.49 m | 06/01/1994 |
| G41/0301 | McNeill Drilling | Existing | ~204.8 (NZ Vertical) | 10.6 m (~194.2) | 28 m | 09/01/2003 |
| G41/0459 | McNeill Drilling | Existing | 202.641 (Dunedin 1958) | 9.86 m (~192.8) | 30.17 m | 17/11/2015 |
| G41/0199 | McNeill Drilling | Existing | 214.72 (Dunedin 1958) | 16.42 m (~198.3) | 27.95 m | 25/11/1999 |
| G41/0004 | Not recorded | Decommissioned | ~198.7 (Dunedin 1958) | Not recorded | 205 m | 1/10/1978 |
| G41/215 | McNeill Drilling | Existing | 212.16 (Dunedin 1958) | 15.47 m (~196.69) | 28.44 m | 05/05/2000 |

Figure 5: Well Locations


Base image sourced from Wells Aotearoa

Lithology records for seven wells are available and are presented as Appendix 1 of this report.

In broad terms, the lithology records indicate that the subsoils below and around the site are primarily comprised of silty and sandy gravel (alluvial or fluvial deposits) extending to approximately 28 m depth below the original ground surface. Glacial deposits or outwash deposits are indicated between depths of 28 m and 124 m, below which are reportedly tertiary lacustrine deposits. Completely weathered Schist rock was reported at a depth of 198 m.

## 4.4 Review of New Zealand Geotechnical Database

We have reviewed the New Zealand Geotechnical Database in preparation of this report, however we did not find additional subsurface investigation records that were relevant to this site.

## 4.5 Review of Related Geotechnical Reports

IE have reviewed the preliminary geotechnical assessment report[12] prepared by Geotago Ltd for the prior plan change application.

Where relevant, the findings and conclusions of that report have been taken into consideration in the preparation of this report.

Geotago's scope of work included a desktop review of publicly available information, and a limited subsurface investigation at site comprised of 14 test pit excavations to a maximum depth of 2.2 m. It should also be noted that the proposed subdivision scheme plan has evolved since Geotago's preliminary report was prepared, and therefore not all the conclusions and recommendations presented by Geotago apply to the current development plans.

Key information from that report is summarised as follows:

- Geotago's assessment identified typical native alluvial deposits as well as areas of uncontrolled fill and pea gravel backfill. The natural ground not affected by the quarry activities is reported as loess overlying native sandy gravels with some cobbles and boulders.

- Geotago have divided the property at 922 Luggate-Cromwell Road into various areas to represent the land use at the time of their assessment (2022) and potential geotechnical constraints relevant to each.
  We have reproduced Table 2 from their report below. A copy of their plan showing the locations of these areas is presented as Appendix 2 of this report:

***Table 2: Site Areas & Potential Constraints***

| ***Site area*** | ***Land Use*** | ***Potential Constraints*** |
|---|---|---|
| *1* | *Administration, plant and equipment* | *Low risk area in terms of ground conditions. Area likely to be maintained as industrial / commercial use*[a] |
| *2* | *Current settling pond* | *This will be backfilled at some point, with residual silt and fine sands having accumulated at the base of the pond. Backfilling likely to be with pea gravels in line with other backfilled areas. Poor ground conditions will likely result from backfilling*[b]*.* |
| *3* | *Operational quarry area* | *Quarry activities have exposed natural gravel sequence. Low risk in terms of ground conditions. Groundwater is shallow* |
| *4* | *Backfilled areas* | *Former quarry areas backfilled with uncontrolled but clean fill. This includes pea gravel to depths of up to 5m. Likely to be geotechnically constrained due to the free-running nature of the pea gravel.* |
| *5* | *Former settling ponds* | *Fill and sediment within the former pond areas are likely to be soft sand and silts, with a high moisture content and low bearing capacity. Remainder of fill will be pea gravels, with their inherent constraints* |
| *6* | *Quarry floor* | *Current exposed natural gravels on the quarry floor where operations have ceased. Groundwater is shallow at this level, but the natural soils have few geotechnical constraints.* |
| *7* | *Southern Batter (Reinstated)* | *This area is the current southern batter and boundary to the site. The former quarry wall has effectively been buttressed with coarse quarry scrapings and pea gravel, mantled with site won topsoil and superficial soils from the quarry extension. The slope is approximately 1V:5H or 11- 12 degrees. The fill is not engineered. Excavation of test pits in this area demonstrated the free running nature of the pea gravels. This would impact on any future earthworks in order to develop the slope area. It is understood that the backfilled area is at a maximum of some 11m.* |
| *8* | *Southern Batter (in process of reinstatement))* | *Pea gravels currently being deposited in this area as part of the quarry reinstatement as per Area 7. Some partial revegetation using quarry scrapings and topsoil for the quarry extension areas. The fill is nonengineered and will present similar constraints to those identified for Area 7.* |
| *9* | *Future quarry Extension* | *Natural ground yet to be quarried. The ground conditions present as the natural sequence of topsoil over loess over sandy gravels. Groundwater is at depth in these locations (i.e. >10m from current ground level).* |

[a] This area is no longer planned to remain zoned Industrial and will be changed to allow residential use.
[b] In this case, Geotago assumed that backfilling will be completed by the quarry operator using uncontrolled pea gravel.

- Broadly, Geotago concluded that there were no significant natural hazards likely to affect the proposed development, with the exception of a minor risk of lateral spreading potentially affecting future waterfront properties and liquefaction affecting the former settling ponds.

- They further concluded that the ground conditions would be generally conducive to the proposed development but noting that specific engineering design "*will be required for the waterfront properties, and the larger commercial, industrial and retail type structures, particularly where these coincide with areas of historical fill*."

- Geotago identified the areas backfilled with pea gravel and the pea gravel slopes as being problematic for the development, and suggested that ground improvement or deep foundations would likely be required for development in those areas.

- Geotago concluded that there were "*no seismic conditions or constraints that negate the development of any building, regardless of its Building Importance Category*". IE interprets this statement as being related to the proximity of the site to the Pisa Fault Zone, noting that the potential for seismic ground shaking exists for the site.

As noted previously, not all of Geotago's conclusions and recommendations are relevant to the current Scheme Plan and development proposals. We also note that some of Geotago's conclusions and recommendations assume that remedial earthworks (or ground improvement) will not be undertaken, for example in their discussions regarding liquefaction potential at prior settling ponds and foundation options at areas of historical fill.

Copies of the relevant site plans from Geotago's report and their test pit logs are presented as Appendix 2 of this report.

## 4.6 Aerial Photograph Review

Limited publicly available aerial and historical photographs reviewed as part of this investigation include:

- Aerial photographs dated between 1958 and 2003 sourced from retrolens.nz[13]; and

- Aerial photographs dated between 2007 and 2021 sourced from Google Earth;

The following table broadly summarises the geotechnically relevant features visible in selected photographs. Photographs with poor resolution or scale were not assessed, and not all details are discussed.

### Table 2: Historical Aerial Photograph Summary

| Date | Description |
|---|---|
| 1958 | The site appears as undeveloped land on the western side of the Clutha River (pre-Lake Dunstan/Te Wairere).<br>There are signs of what appears to be overland flow or meanders of the Park Burn at the site. |
| 1976 | The site appears to be used for agricultural/ crop growing purposes. Flood irrigation appears to have occurred, as well as flooding of the Park Burn. |
| 2001 | The Parkburn Quarry has been established at the northeastern side of the site. The existing offices, concrete and asphalt plants are not yet established, but three structures are visible in the approximate location of the asphalt plant.<br>The topsoil has been stripped, and excavations and stockpiles are apparent near the eastern site boundary, as well as in the area currently containing the asphalt and concrete plants.<br>What appears to be ponded water (~350 $m^2$) is visible near the northeastern boundary. |
| 2003 | The quarry extents are generally similar to 2001 and excavations appear to be ongoing. Ponded water (~5,600 $m^2$) is visible near the northeastern boundary. |
| 2007 | The concrete plant has been established by 2007, and ground clearance activity has occurred at the location of the weighbridge. Quarry activities appear to be ongoing at the northeastern side and areas of ponded water are no longer visible at the northeastern boundary. Stockpiles are visible at the cleanfill area.<br>Operational quarry areas are visible in the approximate centre of the site and extending to the southern boundary.<br>Settling ponds appear to have been formed to the south of the Park Burn outlet. |
| 2010 | The operational quarry areas have extended to the west and east along the southern boundary. |
| 2011 | The weighbridge area is constructed and the asphalt plant also appears to be constructed. Quarry activities are ongoing along the southern boundary. Quarry activities at the northeastern side appear to have ceased. |
| 2015 | Stockpiles are visible at the cleanfill area, and the access road to this area appears dark compared with previous photographs.<br>The extents of the operational quarry area at the southern boundary continue to expand to the west and east. |
| 2017-2019 | Quarry activities continue at the southern side of the site, and ponded water is visible in places in the quarry floor, suggesting that the maximum practical excavation depth has been achieved.<br>The settling ponds to the south of the Park Burn outlet are gradually being filled in during this period.<br>Surficial changes occur around the weighbridge, concrete and asphalt plants over this time, including roading, stockpiling, minor construction etc. |
| 2019-2024 | Quarry activities continue and gradually expand towards the north along the western boundary.<br>By 2021, formation of the pea gravel backfilled slope at the southern boundary has commenced.<br>By 2023, most of the pea gravel slope at the southern boundary has been completed and quarry activities continue northwards. |
| 2024 | There are no significant changes, apart from stripping a portion of land to the north of the concrete plant, dark coloured stockpiles in that area, and changes in vegetation. |

# 5 Preliminary Ground Model

From the results of our desktop review described herein and observations of the site, we infer that the native site soils are generally in keeping with the mapped geology.

The undisturbed greenfield portions of the site are indicated to be covered by a layer of topsoil that is typically 0.2 to 0.3 m thick.

Pleistocene outwash deposits are mapped by GNS for the majority of the site area, and are generally anticipated to underlie most of the site below the surficial Loess/alluvium, as indicated by the well records, Geotago's test pit findings (TP101-107, and TP111-112) and our observations of the quarry. These gravels are anticipated to extend to depths of at least 20 m. Geotago reported that the outwash gravels they identified were typically loose to tightly packed. Therefore, variation in density and consistency may be expected.

Geotago reported encountering a surficial layer of Loess (SILT) at some test pit locations within undisturbed areas. The thickness of Loess ranged from 0.0 to 0.9 m (in TP112) and IE have observed the Loess layer up to 2 m thick in places within the greenfield area during quarry stripping operations. It is possible that the silt may also be the result of alluvial processes, given that GNS have mapped more recent Holocene alluvial/fan deposits to the north of the site.

At the eastern side of the site are areas that have previously been quarried and reportedly backfilled with pea gravel, which is generally a by-product of the quarrying activities. At the cleanfill area, a surficial layer of uncontrolled fill is anticipated to be present over the pea gravel backfill, based on the findings of TP108 by Geotago. This fill is likely to be comprised of variable materials that have been imported to the site over a number of years in a relatively uncontrolled manner, and may contain anthropic and organic inclusions. The thickness of this fill layer is unknown however it is likely to vary over the cleanfill area.

Additionally, from our review of historical aerial photographs, it is possible that some localised areas may have been quarried and backfilled prior to the establishment of the existing concrete and asphalt plants.

The previously quarried areas along the southern boundary and in places on the eastern side of the quarry (south of the Park Burn) have been backfilled with pea gravel, overlying native outwash deposits.

The settling ponds have not been investigated in great detail however we concur with Geotago's assumptions that these are likely to contain soft, silty depositions to varying depths.

The depth and extent of the fill, including imported materials and pea gravel backfill, is not fully quantified as detailed records were not kept, and there are gaps in between historical aerial photographs. Explorations to obtain this information would likely be cost-prohibitive, as well as impractical due to the ongoing quarry activities and scale of the site. Therefore, we conservatively concur with the assumptions made by LandPro when preparing the earthworks plans. Landpro have estimated the thicknesses of the pea gravel and existing fill based on assumed quarry floor elevations of ~195 mRL. Similarly, we have conservatively assumed the "worst-case' extents of various areas of backfill at the site based on historical aerial photographs and discussions with the quarry operators.

The preliminary ground model is summarised on Figure 6 below, however it should be accepted that variations could exist and these may only become apparent once bulk excavations are underway. The three primary areas requiring geotechnical remediation are highlighted in red (Remediation Areas RA1 to RA3).

Groundwater depths based on published information show a static water level between 186 mRL to 198mRL. We expect that groundwater level will on average be in keeping with and approximately conform to lake levels (~194 mRL), although some variations could exist from time to time.

## Figure 6: Summary of Preliminary Ground Model


Base image sourced from Central Otago District Council GIS

# 6 Fieldwork: Gravel Crushing and Compaction Trials

## 6.1 Overview of the Trials

IE was requested to arrange two field compaction trials using crushed pea gravels sourced from Parkburn Quarry. The purpose of the trials was to assess whether or not it was feasible to crush unsuitable pea gravel to create a material suitable for reuse as a bulk structural fill.

To this end, two trials were completed. The first trial was carried out in October 2025 at the Fulton Hogan Miners Road quarry in Christchurch, and used a high pressure grinding roll crusher. A hardfill plateau density test procedure was carried out on a series of trial fill platforms. The trial is fully described in our letter[14] dated 3 November 2025 (reference 24019_1) which is presented as Appendix 3 of this report, and therefore those details will not be reiterated herein.

The second trial was completed at an earthworks site in Wanaka, using pea gravel crushed by a vertical shaft impact (VSI) crusher. The test procedure was generally the same as the first trial, except that the fill platforms were formed in a shallow excavation rather than upon the ground surface. This allowed for survey measurements to estimate the compression/compaction factor of the crushed gravel, as the sides of the fill platforms were confined. Therefore, lateral deformation of the fill platform did not contribute to the compaction factor estimations. A specific summary report for the second trial was not prepared, however the plateau test results are presented as Appendix 4 of this report.

## 6.2 Summary of Results

### 6.2.1 Trial 1

The crushed product was described as a sandy, fine to medium GRAVEL, from the particle size distribution test. Laboratory standard compaction testing carried out on a sample of this material resulted in a Maximum Dry Density (MDD) of 2.06 t/m$^3$ with an optimal water content of 6%.

The MDD resulting from the plateau density trial was 2.07 t/m$^3$ with average moisture content values of 3.1% to 5.3 %.

CBR values of at least 7% were correlated from Impact Hammer testing.

The trial results indicated that crushed pea gravel would be suitable for use as bulk structural fill, and that densities of 95% and 98% of the MDD would likely be able to be achieved during bulk earthworks. The outcome of Trial 1 is discussed in full in Appendix 3.

### 6.2.2 Trial 2

The crushed product was described as a fine gravelly SAND with some silt from the particle size distribution test. Laboratory standard compaction testing carried out on a sample of this material resulted in a Maximum Dry Density (MDD) of 2.06 t/m$^3$ with an optimal water content of 9%.

The MDD resulting from the plateau density trial was 1.96 t/m$^3$. Moisture contents ranged between 2.6 % and 5.1 %.

CBR values of greater than 7% were correlated from Impact Hammer testing.

### 6.2.3 Discussion on Results

Both trials indicated that crushing pea gravel results in an aggregate that is suitable for use as bulk structural fill. Either crushing method is likely to be suitable, noting that slightly differing materials are produced by each machine and production rates are likely to vary.

From Trial 2, the compacted crushed pea gravel was able to be adequately compacted, however the MDD resulting from the plateau testing is around 95% of the laboratory results. It should be noted that the typical water contents of the second trial were approximately half of the optimum water content from the laboratory, and we anticipate that this is a likely contributing factor to the difference. Therefore, a greater application of water during compaction is likely to be required to achieve higher densities.

Fill constructed from crushed pea gravel is indicated to be suitable for supporting future buildings and roads, and further recommendations are presented in Section 8 of this report.

# 7 Discussion on Geotechnical Considerations

The following geotechnical considerations are either mapped or identified for the site as a result of our desk top review and fieldwork. Discussions on the perceived implications are presented below and specific recommendations are presented in Section 8 of this report:

- Seismicity;
- Alluvial Fan and overland flow;
- Existing Uncontrolled Fill and Pea Gravel Backfill;
- Future Earthworks:
    - General Excavations;
    - General Filling;
    - Remediation of Existing Fill Areas;
    - Fill Settlement and Consolidation;
    - Finished gradients and slope angles.

## 7.1 Seismicity

A future significant earthquake is likely to result in moderate to severe ground shaking (as discussed in Section 4.2.1 and 4.2.2 of this report), which may lead to deformations in the ground surface.

The site is mapped by ORC as generally being unlikely to be susceptible to liquefaction, and this agrees with the subsurface materials indicated by the available well records. Geotago have identified that a minor risk of liquefaction exists at the locations of the settling ponds which are assumed to be infilled with soft, moist sediments (from quarrying activities, not natural processes). Additionally, they considered that further specific investigation and design would be needed to assess/mitigate lateral spreading hazard to future waterfront properties, specifically on the peninsulas (which are no longer proposed). Their conclusions were based on a prior scheme plan and did not account for geotechnical remedial works that are being now being proposed to be undertaken.

IE considers that providing that the settling ponds are appropriately remediated such that soft sediments no longer remain, there should be a nil to low risk of earthquake induced liquefaction. This opinion is based on remediation involving removal of all soft sediments from the existing and prior settling ponds (contained within Remediation Area RA2), and backfilling with engineered materials to the design surface elevations.

The native outwash deposits are considered unlikely to be susceptible to liquefaction due to the generally well graded composition of and density of these materials. However, variations cannot be fully discounted, and recommendations are presented in Section 8.1 if loose materials are encountered close to the lake elevations during the earthworks. In general terms, structures sited upon competent, unsaturated soils at least 3 m[15] above potentially liquefiable materials are considered to be unlikely to experience damage from liquefaction ejecta. At this site, future ground surface elevations are shown as at least 4 m above the lake level.

Further, any land adjacent to future coves is likely to be constructed of engineered fill, which may be reinforced (by retaining or incorporation of geogrids), if necessary, to mitigate potential lateral displacements that may result from a seismic event. Such areas will be subject to specific engineering design and further discussion and recommendations are presented in Sections 7.4.5 and 8.

The site is further located sufficiently far from the mapped Pisa Fault Zone such that fault rupture is unlikely to be a consideration for this proposed development, in accordance with the report[16] prepared by GNS for the Ministry for the Environment for planning land development close to active faults.

## 7.2 Alluvial Fan and Overland Flows

Part of the site including the Park Burn is mapped as active alluvial fan (floodwater dominated, refer to Figure 4 of this report). Detailed assessment of the Park Burn flooding potential and active fan effects are outside of the scope of this report..

Further specific assessment should be carried out by a suitably qualified consultant to assess potential flooding of the Park Burn Creek and effects on the proposed development.

## 7.3 Existing Uncontrolled Fill and Pea Gravel Backfill

Three primary areas containing existing uncontrolled fill and pea gravel backfill have been identified at the site (Remediation Areas RA1 to RA3). These areas are outlined in red on Figure 6 of this report. Additionally, the area highlighted in yellow on Figure 6 potentially also contains localised areas of existing uncontrolled fill.

Potential issues associated with the existing fill and pea gravel backfill include unstable building areas leading to differential settlement below future buildings, unstable slopes in a future seismic event, ongoing consolidation settlement in areas containing wet, soft sediments (settling ponds). The filled and pea gravel areas are unsuitable for future development in their current condition. Therefore, mitigation measures are required to be implemented.

The proposed mitigation measures involve remediation of these areas by:

- Over-excavation of existing fill;
- Sorting and disposal of unsuitable materials;
- Crushing of pea gravel; and
- Replacement of uncontrolled fill with engineered, crushed materials or local borrow materials to the design surface elevation.

Further discussion is presented in Section 7.4.3 below, and in Section 8 of this report.

## 7.4 Future Earthworks

From the earthworks plans, we understand that all of the site area will be affected by future earthworks, involving either excavation or filling (or both in the remediation areas) to achieve the design subgrade elevations. Therefore, actual foundation bearing and other geotechnical design parameters (e.g. for retaining wall and pavement design) would be assessed upon completion of the earthworks.

### 7.4.1 General Excavations

Maximum excavations heights are indicated to be up to 15 m, and are generally expected to expose native materials like those visible in the quarry sides. Native deposits could include surficial Loess, layers and lenses of silts, and fine to coarse sands and gravels that are typical of Pleistocene outwash deposits.

The subgrade conditions exposed by the excavations could be variable (in density and type), subject to the actual materials present.

### 7.4.2 General Filling and Local Borrow Materials

Fill using local borrow materials is indicated to a maximum depth of around 12 m. No overly steep fill batters are proposed within the site boundaries, however if required consideration can be given to retaining or mechanically stabilised earth (MSE) structures.

Fill is generally expected to be relatively straightforward as most of the borrow materials are anticipated to comprise silty and sandy GRAVEL, however based on our knowledge of the locality, some of the following challenges may be experienced:

- Variable, weak or loose subgrade conditions in places may lead to challenges with compaction of the fill;
- Earthworks adjacent to the Park Burn Creek will require specific erosion and sediment controls; and
- Engineering quality control of potentially variable fill materials.

These challenges should be able to be mitigated with normal engineering practices and management.

### 7.4.3 Remediation of Existing Filled Areas

Geotechnical remediation of the existing filled areas should be managed separately from the general earthworks. As discussed in a preceding section the remedial works are likely to involve:

- Over-excavation of existing fill;
- Sorting and disposal of unsuitable materials;
- Crushing of pea gravel; and
- Replacement of uncontrolled fill with engineered, crushed materials or local borrow materials to the design surface elevation.

Geotechnical remediation should not be required in areas that do not contain existing fill or pea gravel backfill. Therefore, the actual extents of remediation may be less than indicated by the earthworks plans and this report.

Geotechnical challenges that might be encountered during the remediation work may include compaction of crushed materials near the groundwater surface (subject to the depth of over-excavation), quality control (including water content) and logistical matters, however these should be able to be mitigated with normal engineering practices and management. Deep fill embankments should be well constructed to mitigate settlement of fill due to self weight, and potential differential settlement between filled and unfilled areas (see Section 7.4.4 below).

At the cleanfill area and the existing concrete and asphalt plants (shown in orange and yellow on Figure 6) the geotechnical remediation is required in conjunction with environmental/contamination controls, and therefore materials from this part of the site should be kept separate from other areas. Specific remedial measures in this area are planned such that the resulting ground surface is suitable from both geotechnical and environmental perspectives. Reference should be made to Section 5.2 of the Remediation Action Plan[17], which presents the description of the environmental/contamination remedial works. We have reproduced the following text from that document for clarity on how the geotechnical objectives will be achieved during these works:

- *“Site establishment and kick-off meeting;*
- *Excavation and stockpiling of impacted material from the areas identified by e3[1];*
- *Confirmation of the significance of contamination impacts in material from the asphalt plant, fuel storage locations and former orchard area through laboratory testing;*
- *Screening of the ‘cleanfill’ material to remove large inclusions;*
- *Re-placement and compaction of the ‘cleanfill’ and marginally impacted material from other parts of the site, once the underlying pea gravels have been excavated from the north eastern corner of the site;*
- *Capping the ‘cleanfill’ material with at least 2 m of clean material* [or crushed pea gravel] *from other parts of the site;”*

### 7.4.4 Fill Settlement and Consolidation

From NZS4431:2022:

> *“One of the most important factors in ensuring satisfactory performance of stable fills is the limiting of post-construction differential settlements. Acceptable differential settlements should be identified and documented.”*

Settlement considerations include settlement within in the fill itself and (consolidation) settlement of the native materials below the fill. Additionally, fill may also settle differentially to any adjacent non-filled or excavated areas.

Soils that are susceptible to consolidation settlement typically include normally consolidated fine-grained soils (or otherwise compressible materials) and any soil type that experiences a slow release of excess pore water pressure generated from the placement of new loads.

Post construction settlement of the fill itself may occur due to self weight or poor construction methods. Additionally, collapse compression is defined by NZS4431 as:

> “*Settlements developing in partially saturated and low to medium dense soils because of fluid infiltration. Collapse compression settlements can be rapid or develop over an extended period.*”

Generally, well constructed, well graded fills with appropriate drainage and stormwater management should be at relatively low risk of collapse compression. Settlement of granular materials tends to occur over relatively short periods and should be largely expressed during earthworks or construction. Fine grained type soils may experience settlement over longer periods of time.

Monitoring of settlement during and post construction of the fills is a measure that will assist in evaluating the total and differential settlement, as well as timeframes over which settlement occurs.

### 7.4.5 Finished Gradients and Slope Angles

The finished surface gradients are depicted on a plan prepared by LandPro (reference 114_01)

Most of the site area is generally indicated to be very gently sloping (< 10 %) with no overly steep excavation batter angles proposed.

A 7 m tall and 50 m wide slope battered at between 16 and 20 % (up to 1(V):5(H)) is proposed at the western side of the site. This slope is to be formed by excavation of the native outwash gravels. Preliminary slope stability analysis was completed for this slope. The results indicate acceptable factors of safety under assumed normal, static conditions (minimum FoS ~2.8). Under modelled seismic conditions, resulting yield accelerations range between 0.3 and 0.4 g, suggesting that some slope movement is possible in a 1 in 500 year seismic event for Cromwell (where the design peak ground acceleration for a 1 in 500 year event is 0.35g). Calculated deformations for this scenario do not exceed 10 mm.

At the eastern side and adjacent to the proposed coves, the ground surface is generally around 4 m above the lake level with slope angles ranging between 20 and 50 % (up to 1(V):2(H)). Some slopes adjacent to the lake edge are existing, and some will be engineered during the earthworks. Slope stability analyses have not been carried out for these slopes yet. For slopes to be constructed by engineered filling, we anticipate that future measures such as slope stability analysis would be carried out leading to any required mitigation measures. These may include fill reinforcement (e.g. with geogrids or retaining walls) if necessary, appropriate set back distances and specific foundation design as necessary may be applied to protect future structures and infrastructure. Existing slopes are likely to be managed by either re-grading if possible or applying set-back distances.

The coves are generally understood to be formed with 33% (1(V):3(H)) slopes below the water line (in native alluvium). The northern cove is proposed to be 4 m deep and the southern cove is proposed be no greater than 1.5 m deep. Such slope gradients are generally acceptable for permanent unsupported batters in the local materials. Additionally, edge protection involving rock liners/reno mattresses, gabions etc are proposed for the sides of the coves, which should be suitable based on similar conditions at Pisa Moorings to the south of the site. It is our further understanding that retaining walls (likely to be mass block or similar) are proposed for the southern edge of the Northern Cove as a minimum.

# 8 Conclusions and Geotechnical Recommendations

## 8.1 General

Based on the information presented herein and our knowledge of the area, it is our opinion that the site is geotechnically suitable for future residential development, subject to the recommendations of this report.

From the prior subsurface investigations completed by Geotago and our knowledge of the locality, we anticipate that the native subsurface conditions should be generally in keeping with published mapping.

Areas containing existing fill (pea gravel backfill and imported cleanfill) have been broadly quantified based on assumed worst case conditions. This approach has been taken to ensure that the geotechnical remediation can be appropriately budgeted, managed and implemented. However, it is possible that other areas containing existing fill have not yet been identified. The Geotechnical Engineer should be consulted in the event that such areas are identified during the earthworks, so that appropriate remediation measures can be extended to these areas as well. Geotechnical and environmental remediation is required to be carried out concurrently in the cleanfill areas, as well as potentially at the areas around the existing concrete and asphalt plants.

Potential seismic effects at the site could include moderate to severe ground shaking (as discussed in Section 4.2 of this report), which may lead to deformations in the ground surface. Appropriate engineering design of structures and underground services, as well as appropriate construction of cut and fill embankments (where relevant). The site is not anticipated to be subject to earthquake induced liquefaction based on the subsurface conditions indicated by the well records, and providing that settling ponds etc. are appropriately remediated to mitigate any such potential. However, if loose, sandy deposits are encountered close to or below the lake level then the liquefaction and lateral spreading potential should be reassessed so that appropriate mitigation measures can be implemented. Fault rupture of mapped faults is considered unlikely to adversely affect the site.

Assessment of alluvial fan activity and flooding that may result from the Park Burn Creek are beyond the scope of this report. We understand that an assessment has been carried out for the neighbouring property to the north and the results may be applicable to the site. If the outcome of that assessment indicates that the proposed development could be adversely affected by potential flooding of the Park Burn Creek, then further specific assessment to inform appropriate mitigation measures may be required.

Groundwater levels are likely to conform to the existing lake levels, and should be taken into account if geotechnical remediation works are likely to come close to or contact groundwater. Methods to manage and control water inflow during excavations for the coves should be allowed for.

## 8.2 Bulk Earthworks

### 8.2.1 General

Bulk earthworks should be undertaken in accordance with NZS4431:2022 Engineered Fill for Lightweight Structures, Good Practice Guidelines for Excavation Safety published by Worksafe (July 2016) and the recommendations of the Geotechnical Engineer.

Remedial earthworks in the “cleanfill’ areas should be carried out in conjunction with the recommendations of the Geotechnical Engineer and a Suitably Qualified Environmental Practitioner (SQEP).

It is the responsibility of the contractor to ensure that all earthworks are undertaken in accordance with the relevant Health and Safety legislation, implement environmental and erosion controls and to protect neighbouring properties from adverse impacts potentially resulting from the earthworks.

The following sections provide geotechnical recommendations in addition to those presented in NZS4431 which are specific to the proposed earthworks.

#### 8.2.2 Site Preparation

Generally, fill areas should be prepared by stripping all topsoil, vegetation, loose surficial materials, existing fill and overly coarse materials (e.g. boulders) where present to expose competent native deposits.

Similarly, excavation areas should be cleared of unsuitable materials to expose acceptable borrow materials for use as bulk fill.

Remediation areas should be prepared by excavating all existing fill and pea gravel and compacting the exposed subgrade if necessary, prior to replacement with engineered fill materials. Excavations adjacent to site boundaries should be controlled/staged such that neighbouring properties are not adversely affected.

Topsoil stockpiles should be sited well clear of the works to avoid contamination with bulk fill materials. Stockpiles of unsuitable materials should be sited well clear of the works on stable areas or disposed in an appropriate manner off site.

If any areas of weaker soil are identified during the site clearance activities, then further localised excavation and backfilling may be required. Advice should be sought from the Geotechnical Engineer in the event that this work is necessary.

The Geotechnical Engineer shall view all stripped areas and undertake testing as necessary to verify the stiffness of the subgrade prior to placement of new fill materials.

#### 8.2.3 Fill Materials

Fill specifications should be prepared for all the materials to be used as fill at the site. Draft specifications are presented as Appendix 5 of this report, however these should be reviewed prior to works commencing subject to actual materials used.

Fill materials are generally expected to be comprised of either local borrow materials (Loess, and sandy GRAVEL alluvium/outwash deposits) or re-purposed pea gravel that is crushed to form a suitable fill material.

##### Borrow Materials

Borrowed fill materials may vary, and further assessment of these soils may be required to inform compaction specifications. From our limited observations, the materials identified to date at the site should be able to be used as engineered fill subject to the appropriate controls and methods being implemented for the various soil types. Further field compaction trials, such as plateau testing, may be carried out on local borrow materials to further inform the fill specification and compaction methodologies, and to assess the actual performance of the site materials. Where a material change is identified, additional acceptance testing shall be carried out as per NZS4431.

Boulders (typically particles in excess of 200 mm length) should ideally be removed from the fill materials.

#### Crushed Pea Gravel

IE have observed two compaction trials using crushed pea gravel specifically to inform the geotechnical remediation strategies to be implemented at Parkburn. It is our opinion that crushed pea gravel should be suitable to be used as engineered fill providing it is crushed with appropriate machinery, conditioned with respect to water content and compacted with suitably heavy, vibrating plant.

The crushed gravel is likely to be produced as a uniform, consistent product which allows for more efficient on-site quality controls during bulk earthworks. The addition of water must be allowed for, both for compaction and for dust control.

#### Other Materials

Other existing fill materials that may be proposed to be re-used as engineered fill (e.g. crushed concrete, other aggregates etc.) will be subject to approval by the Geotechnical Engineer and potentially also a SQEP.

Any existing fill materials are that deemed to be unsuitable for re-use as engineered fill (e.g. due to excessive unsuitable inclusions, contaminants or compressible materials) should be disposed of in an appropriate manner and not be incorporated into the engineered fill.

### 8.2.4 Fill Stabilisation and Sloping Ground

Some areas may require fill compaction upon weak/unfavourable native subgrades. Specific design may be required for these areas, involving either earthworks methods or incorporation of geotextiles and grids which may assist to stabilise unfavourable subgrades, such that the design requirements can be achieved. These areas should be assessed and mitigated once identified (should they exist).

Fill placed on sloping ground steeper than 1(V):4(H) should be appropriately keyed and benched into the slope to avoid downward migration of fill materials. Typical depths for shear keys and bench heights are between 0.6 m and 1 m, however this may vary slightly in different parts of the site. The width of the shear keys and benches will be determined by the size of machinery on site, and the quantity of the benches will be dependent on the actual slope geometry.

In broad terms, fill embankments with batter slopes steeper than 1(V):3(H) may require stabilisation, subject to the type of fill materials, height of the batter slopes and proposed development near to the batter. Stability of the proposed filled batters should be assessed and appropriate retaining structures designed as necessary. Mechanically stabilised earthfill using geogrid reinforcement may be considered as an alternative to retaining structures, if necessary.

### 8.2.5 Fill Placement and Compaction Testing

Fill placement and compaction methods for local borrow materials should be confirmed prior to bulk earthworks commencing, by conducting a fill compaction trial. Compaction methods for the crushed pea gravel have been largely determined by the trials completed to date, and will be verified once the actual crushing machinery etc. is confirmed.

Field compaction testing should be carried out by Nuclear Density testing (as per the specification) to verify that the appropriate degree of compaction has been achieved. A minimum target dry density of 95% of the maximum dry density (MDD) is typically recommended for normal residential development. However, in areas where the height of fill exceeds 3 m at this site, we recommend a minimum target dry density of 98% of the MDD should be aimed for. This recommendation may be reviewed subject to the performance of actual materials used and outcomes of further field trials.

Supplementary testing using a Scala penetrometer and/or Impact Hammer may be carried out as directed or by the Geotechnical Engineer. If continuous compaction control methods are used then field compaction testing as above should be undertaken periodically to verify results.

Field compaction testing shall be carried out by an independent testing company or the Geotechnical Engineer. Test results shall be forwarded to the Geotechnical Engineer (when completed by others) for review as soon as completed and on an ongoing basis during the earthworks programme.

If any tests indicate failure to comply with the earthworks specification, then the contractor is required to rework the area in question as necessary to achieve compliance. The reworked area shall be retested and return a satisfactory result before further fill placement can continue.

#### 8.2.6 Fill Settlement

Fill settlement due to self-weight could lead to differential settlement occurring between adjacent filled and excavated areas, as well as within areas of deep fill itself.

Settlement of fill due to self weight is a potentially complex issue that could affect fill greater than 6 m thick. Estimated settlements could range between 0.5 and 2 % of the total fill thickness[18], subject to the quality and type of the fill, and time taken to construct. Fill constructed of primary granular materials, such as is the case for this site, is most likely to experience immediate settlement during a relatively short time frame. Long term, creeping settlements are usually anticipated for cohesive clays and silts, and not for the site materials, however creeping settlement due to self weight cannot be discounted given the proposed fill heights of up to 12 m.

Mitigation methods include specific construction methods and timing for such fill embankments to ensure high quality of earthworks, appropriate benching, and a sufficient period of settlement monitoring of completed areas. Subject to the outcomes of these methods, additional methods such as temporary over-filling/pre-loading to induce most settlements could be carried out prior to construction of permanent structures and pavements.

#### 8.2.7 Excavations and Batter Slope Angles

Excavation safety on site is responsibility of the contractor, and the recommendations of Worksafe should apply.

Permanent, unsupported excavation and fill batter slopes should not be formed any steeper than 1(V):2(H). As discussed in Section 8.2.4 above, batter slopes steeper than 1(V):3(H) may require stabilisation, subject to the type of fill materials, height of the batter slopes and proposed development near to the batter. Stability of the proposed batters should be assessed and appropriate retaining structures designed as necessary.

Temporary batters may be formed at steeper angles subject to approval by the Geotechnical Engineer, depending on the height of the temporary slope and other work area conditions. This is best addressed on site at the time of construction. The Geotechnical Engineer should be contacted immediately if any temporary or permanent slopes display signs of instability, particularly if any personnel, neighbouring properties, infrastructure or other assets are potentially at risk of harm or damage.

#### 8.2.8 Erosion Controls

Fine grained materials, such as the surficial silt (Loess) identified at the undisturbed parts of the site, are potentially dispersive and subject to erosion when exposed to water, and dust generation in windy conditions. Crushing operations are also likely to produce fine grained materials that will form sediment and dust.

Appropriate controls should be implemented during bulk earthworks to minimise these effects. The design of short term erosion, sediment and dust control during earthworks is undertaken by others and is excluded from this report.

Permanent measures such as vegetating, hydro-seeding and/or the placement of proprietary erosion control mats, should be carried out to stabilise the surface of any unsupported batters and bunds to protect against long term erosion and the formation of dust. This recommendation may also apply to temporary batters, where the batters are to remain exposed for prolonged periods of time (i.e. months rather than weeks).

### 8.2.9 Formation of Coves

The proposed coves are likely to be formed by excavation and filling to form the batters adjacent to the waterways, once the temporary stormwater and sediment works are completed. Therefore, we anticipate that cove formation will be towards the end of the earthworks programme.

Excavation below ~195 mRL to achieve the design depth below the water line will require management of water inflows. We anticipate that water inflow management will be by sump pumping methods. Static excavation will likely be required if water inflow methods become unachievable.

The sides of the coves are proposed to be battered at 1(V):3(H), and edge protection to reduce erosion is likely to be by means of rock armour.

## 8.3 Settlement Monitoring

Monitoring of completed filled areas using accurate survey measurements is recommended to be carried out.

Monitoring should continue until a trend in the data shows that primary settlements have been expressed, and that creeping settlements are either non-existent or negligible. Further it is recommended that areas containing the maximum height of fill should be constructed first, so that adequate monitoring periods can be allowed for in the development programme.

## 8.4 Preliminary Retaining Structures

The following broad recommendations may apply for the design of retaining structures, however we recommend that designers verify the geotechnical conditions and type of support prior to designing any structures with the Geotechnical Engineer.

### 8.4.1 General Design Considerations

The designer should allow for any other relevant surcharges that are likely to be imposed on the wall, including any relevant building, vehicle, backslope, toeslope and seismic loads.

Depending on the style of wall, toe drainage is likely to be required. Toe drainage should be connected into an approved stormwater disposal system.

We anticipate that the retained and foundation soils around future retaining walls will be engineered local materials or crushed pea gravel fill. The following moderately conservative geotechnical parameters are assessed for typical sandy gravel materials:

- Cohesion $c'$ 0 kPa
- Internal friction $\varphi'$ 32 degrees
- Unit weight $\gamma$ 19 kN/m$^3$

### 8.4.2 Seismic Design Considerations

Reference should be made to the Ministry of Business, Innovation and Employment (MBIE) / New Zealand Geotechnical Society (NZGS) Earthquake Resistant Retaining Wall Design (Module 6 version 1) guidance issued in November 2021.

Seismic design calculations should be carried out for any walls that meet the criteria outlined in Module 6.

The site may be taken as Class IV and the topographic amplification factor may be taken as 1.0 in this case. The appropriate wall displacement factor ($W_d$) should be selected from Table 5.2 of Module 6.

The maximum peak ground acceleration (PGA) for various return periods for Cromwell is given in NZS1170.5:2025 Table 3.1 (part 49).

## 8.5 Future Subgrade, Foundation and Set Back Conditions

Final foundation recommendations for future dwellings are subject to developed/final design plans and the actual extent of earthworks. Factors to consider include excavation and fill depths, successful bulk earthworks construction, potential for settlement, proximity to slopes and material types.

Based on the findings of Geotago's preliminary investigations, the compaction trials completed to date and our understanding of the proposed earthworks, we anticipate the site conditions to be generally suitable for shallow foundation types for most light weight residential buildings and specifically designed commercial buildings, following successful bulk earthworks and remediation of currently unsuitable areas.

Any filled areas requiring specific design should be identified and documented upon completion of the earthworks. Similarly, excavated areas will need to be assessed following completion of earthworks such that appropriate foundation and pavement design recommendations can be made.

Any lots with unsupported batter slopes or retaining walls on the boundaries may be subject to foundation set-back distances. Set back distances should be applied to the crest of unsupported slopes, top of retaining walls and from the base of retaining walls to allow access for any future maintenance that may be required.

## 8.6 Preliminary Pavement Design CBR

Based on the field compaction trials completed to date, we consider that preliminary pavement design may be based on a CBR of at least 7, however some variation should be expected given the scale of the development.

Some stabilisation measures or specific pavement design may be needed where the final CBR values are lower than 7. These measures may involve additional compaction, aggregate or the use of geotextiles and/or geogrids incorporated into the roading basecourse and are best assessed on site during the earthworks programme.

## 8.7 Stormwater and Surface Water Management

Finished platform surfaces should be positively graded with appropriate slopes at all times to provide for efficient removal of surface water runoff and discharge to the approved stormwater disposal system. Therefore, finished grades should be sustained after completion of earthworks and areas potentially susceptible to minor slumping should be finished at slightly steeper gradients (such as 2.5 to 5%)

For future building reference, ponding of water should not be permitted near or under the future building areas, as well as foundation excavations and filled batters, at any time during or after construction. As a guide, surface gradients of 1 in 25 (4%) for a minimum distance of 1 m around the building perimeter are specified in NZS3604 for buildings with concrete slab foundations.

Additionally, to prevent surface runoff from entering building areas, perimeter berms or swales may be constructed as necessary at the top and bottom of slopes or across depressions to re-direct surface water into approved drainage systems.

## 8.8 Landscaping Areas

Landscaping areas that are proposed to be vegetated with trees etc., may require a different approach with respect to earthworks to facilitate proper growth and allow the survival of plants, as tightly compacted crushed gravel may not provide optimal growing conditions.

Therefore, consideration should be given to a more relaxed earthworks specification for the surficial layers at such areas.

## 8.9 Further Assessments and Reporting

Any areas that are identified as requiring further geotechnical assessment during the earthworks programme should be assessed by the Geotechnical Engineer of record prior to issuance of S223/S224 certificates so that the appropriate scope of remedial work can be determined and implemented. Examples may potentially include, but are not limited to, areas with finished gradients greater than 1(V):3(H), areas requiring retaining walls that are likely to extend beyond single lot boundaries, areas containing loose, saturated silts and sands and areas containing previously unidentified existing fill.

Future lot specific geotechnical information is to be presented in a series of Geotechnical Completion Reports (publication of completion reports is likely to be staged in accordance with the earthworks stages). Any areas requiring further geotechnical assessment upon completion of earthworks will also be identified in these reports, as well as other lot specific geotechnical constraints, such as set back distances, retaining requirements, foundation types, CBR etc.

# 9 References


1. Turnbull, I.M 2000. Geology of the Wakatipu area. Institute of Geological and Nuclear Sciences1:250 000 geological map 18. 1 sheet plus 72 p. Lower Hutt, New Zealand: Institute of Geological and Nuclear Sciences Ltd.

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# 10 Limitations

i. We have prepared this report in accordance with the brief as provided. This report has been prepared for the use of our client, Fulton Hogan Land Development Ltd, their professional advisers and the relevant Territorial Authorities in relation to the specified project brief described in this report. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity.

ii. The recommendations in this report are based on the ground conditions indicated from published sources, site assessments and subsurface investigations described in this report based on accepted normal methods of site investigations. Only a limited amount of information has been collected to meet the specific financial and technical requirements of the client's brief and this report does not purport to completely describe all the site characteristics and properties. The nature and continuity of the ground between test locations has been inferred using experience and judgement and it should be appreciated that actual conditions could vary from the assumed model.

iii. Subsurface conditions relevant to construction works should be assessed by contractors who can make their own interpretation of the factual data provided. They should perform any additional tests as necessary for their own purposes.

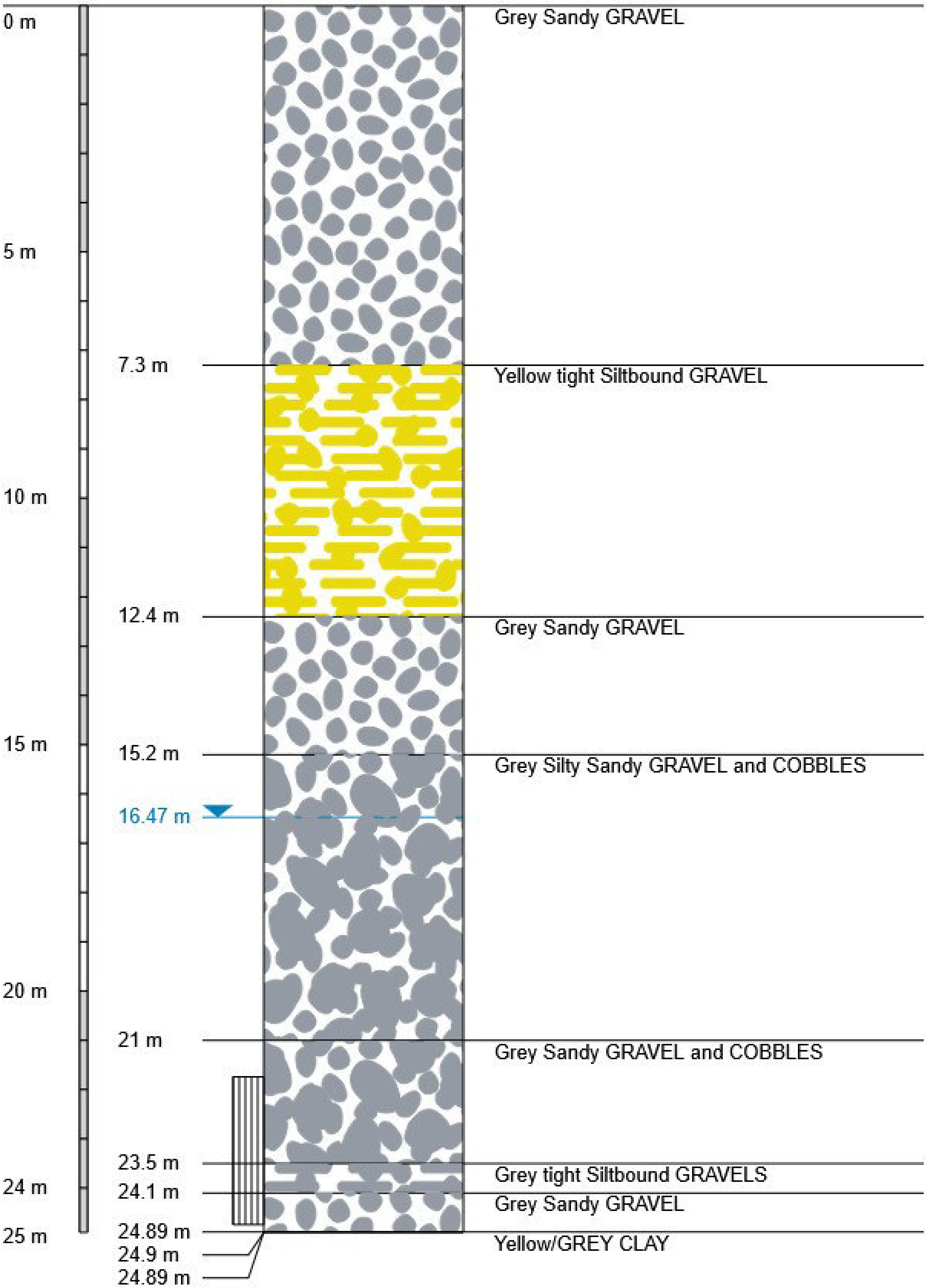
iv. This Limitation should be read in conjunction with the IPENZ/ACENZ Standard Terms of Engagement.

v. This report is not to be reproduced either wholly or in part without our prior written permission.

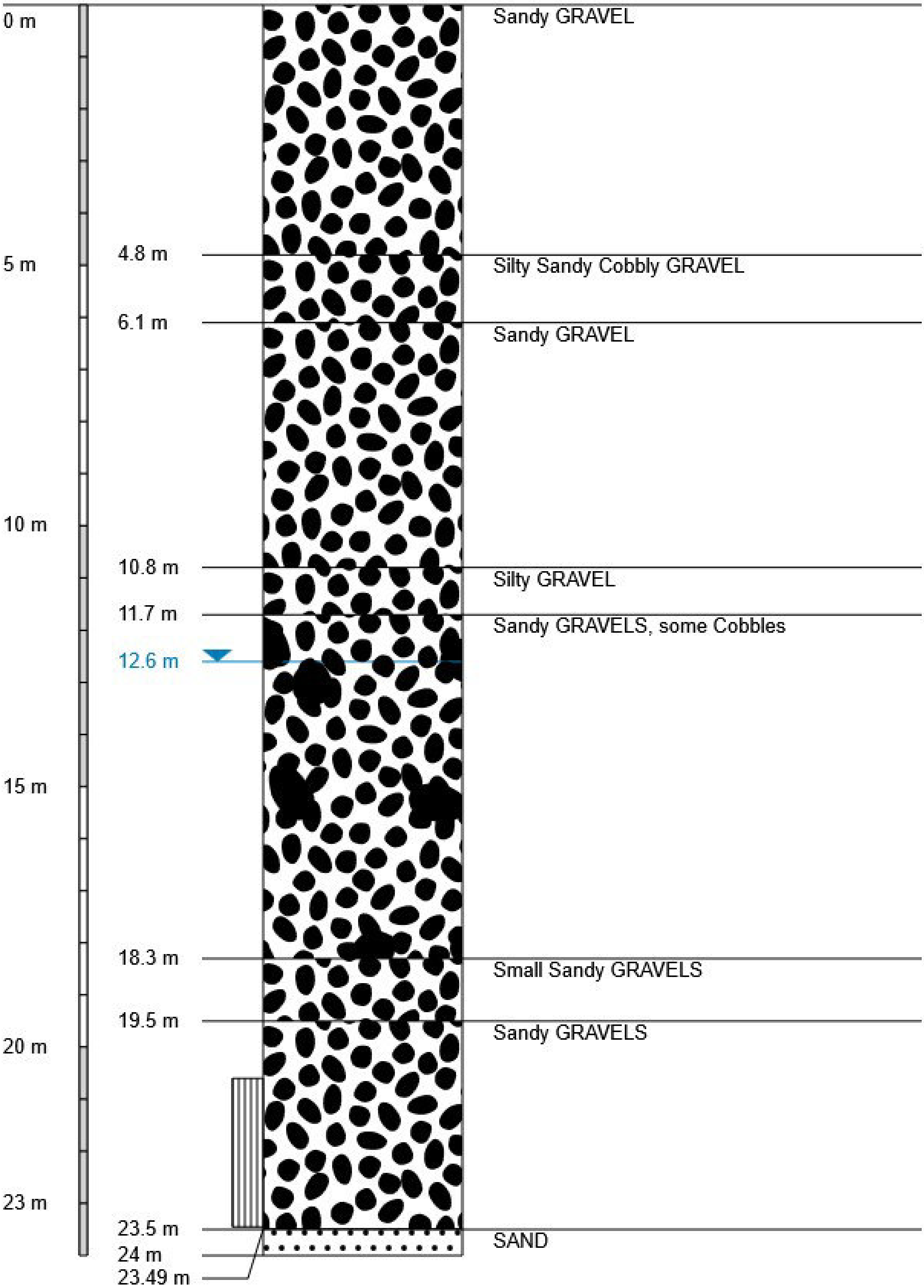
# Appendix 1

## Well Lithology Records

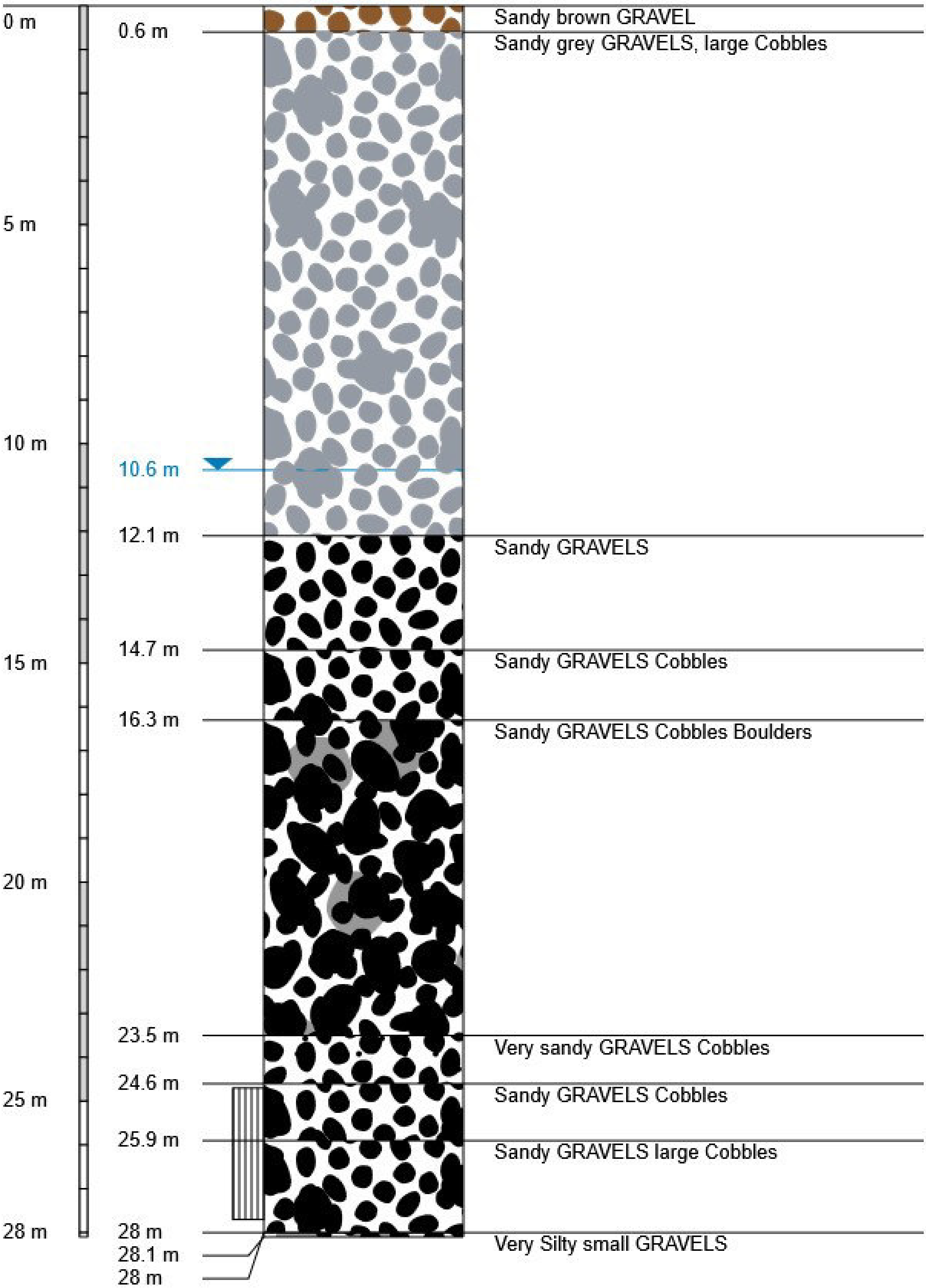
**Council well number** : G41/0113
**Well name** : -
**Drilling company** : McNeill Drilling
**Drilling date** : 12/08/1992
**Drilling method** : Rotary/Percussion
**Locality** : -
**Total depth drilled** : 24.89m
**NZTM** : 1304746 : 5014765



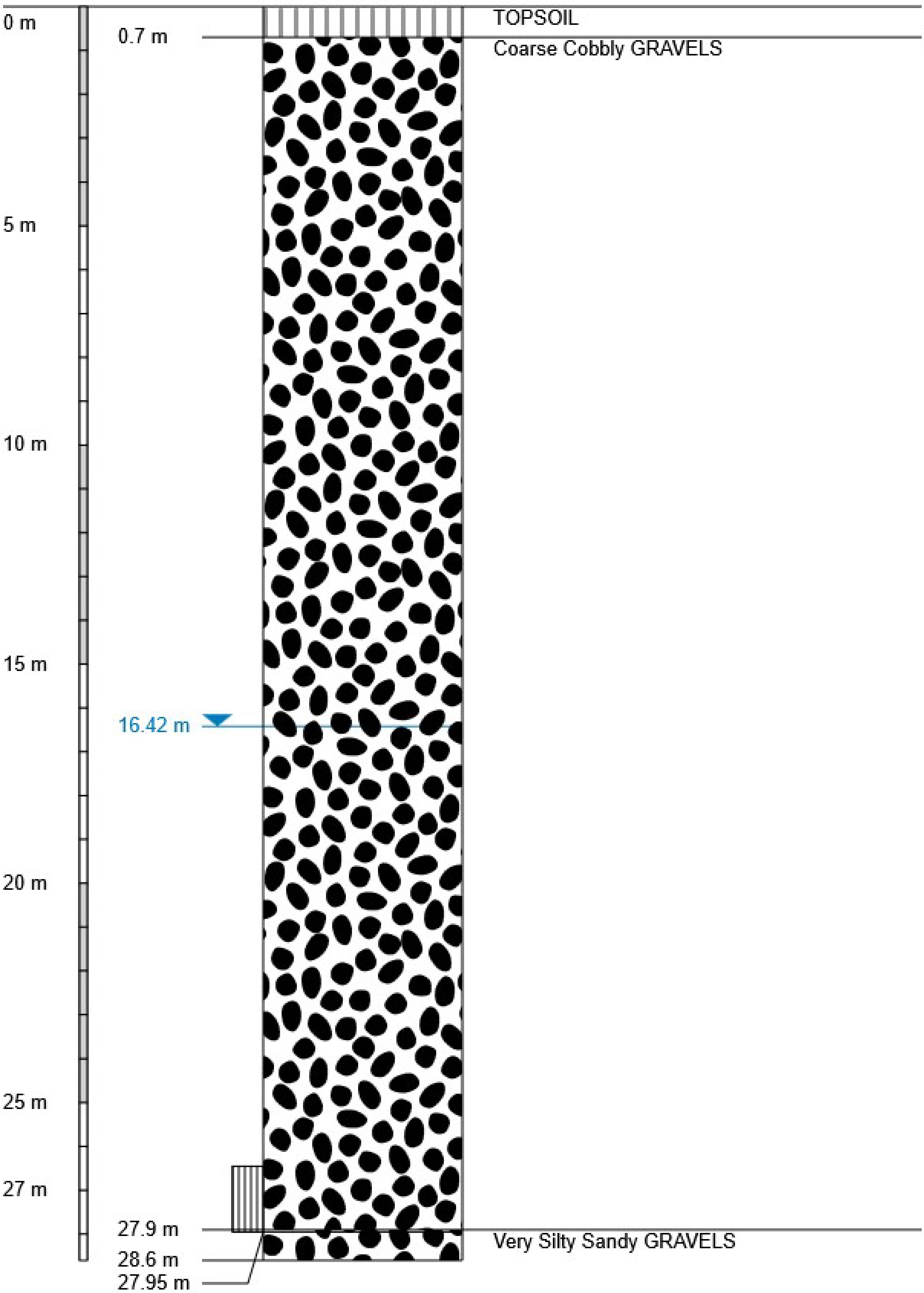
**Council well number** : G41/0115
**Well name** : -
**Drilling company** : McNeill Drilling
**Drilling date** : 06/01/1994
**Drilling method** : (unknown)
**Locality** : -
**Total depth drilled** : 23.49m
**NZTM** : 1303935 : 5014058



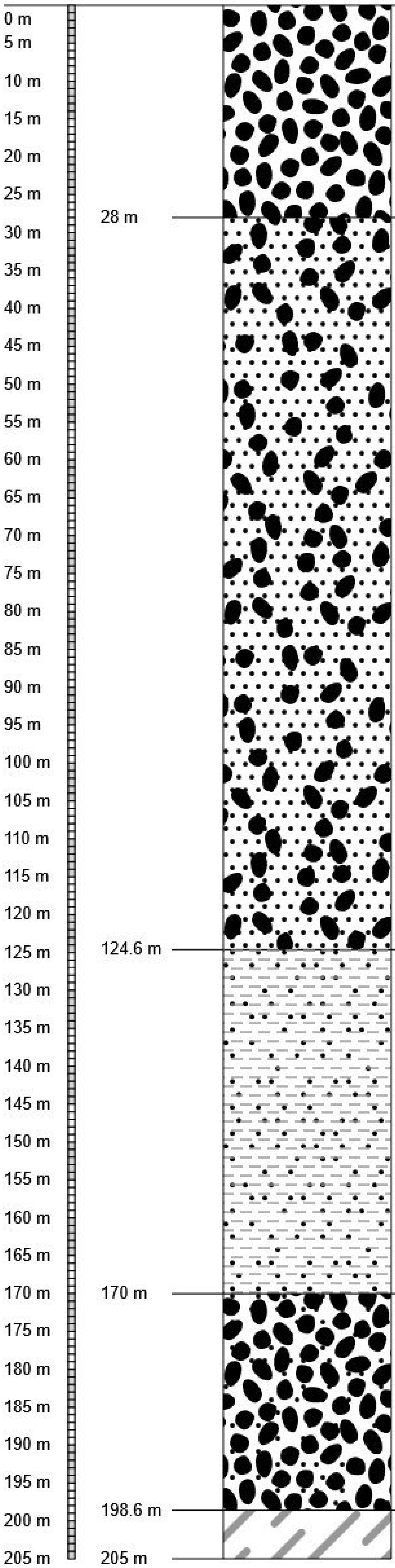
**Council well number** : G41/0301
**Well name** : -
**Drilling company** : McNeill Drilling
**Drilling date** : 09/01/2003
**Drilling method** : Cable Tool
**Locality** : Mount Pisa
**Total depth drilled** : 28m
**NZTM** : 1304283.33 : 5013954.88



**Council well number** : G41/0199
**Well name** : -
**Drilling company** : McNeill Drilling
**Drilling date** : 25/11/1999
**Drilling method** : (unknown)
**Locality** : -
**Total depth drilled** : 27.95m
**NZTM** : 1303799.63 : 5014325.75

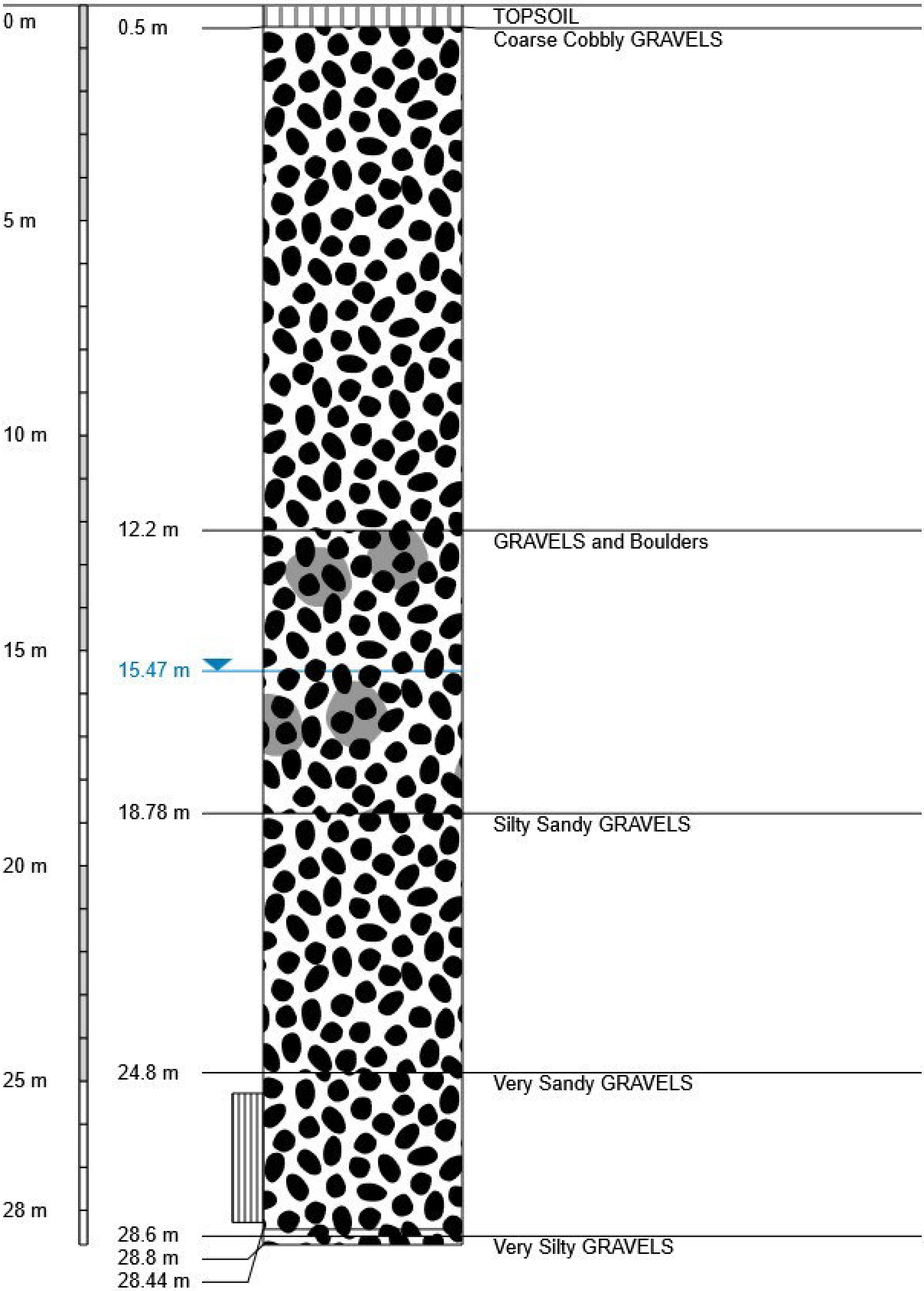


**Council well number** : G41/0004
**Well name** : -
**Drilling company** : -
**Drilling date** : 01/10/1978
**Drilling method** : (unknown)
**Locality** : -
**Total depth drilled** : 205m
**NZTM** : 1304801.37 : 5014369.66

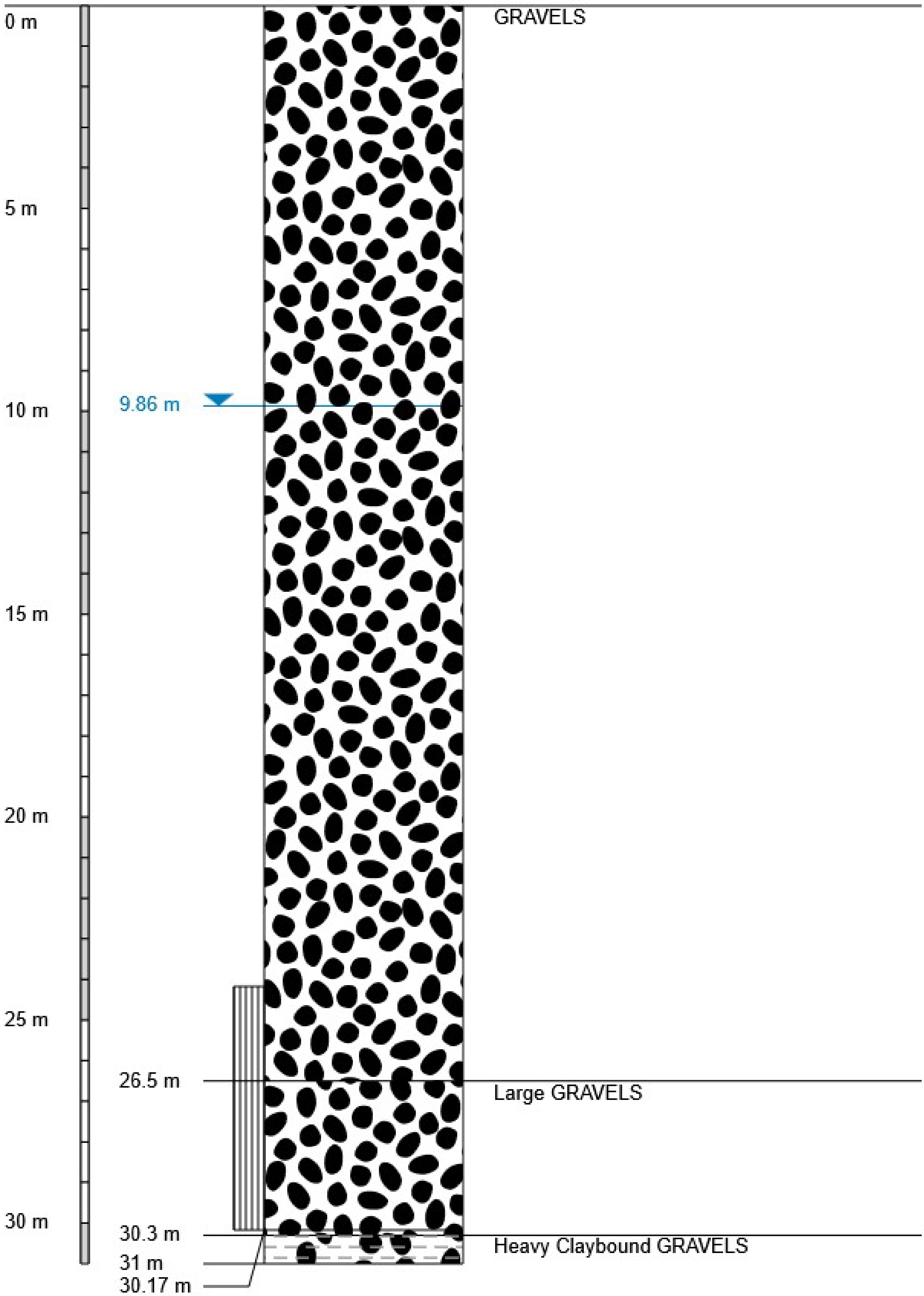



| | |
|---|---|
| | Alluvium, fluvioglacial, horizontally bedded |
| 28 m | Glacial lake sediments: Predominantly SILT and SAND with dropped Pebbles, Gravelly, non-coherent SAND near base, no definite Till but firmer near base |
| 124.6 m | Tertiary lacustrine sediments: Sandy mottled CLAY with Silt and rare beds of fine Quartz Sand |
| 170 m | Tertiary lacustrine sediments: Coarsening downwards; Clayey Schists Quartz GRAVEL and SILT to medium SAND |
| 198.6 m | SCHIST, completely weathered |
| 205 m | |

**Council well number** : G41/0215
**Well name** : -
**Drilling company** : McNeill Drilling
**Drilling date** : 05/05/2000
**Drilling method** : (unknown)
**Locality** : -
**Total depth drilled** : 28.44m
**NZTM** : 1303615.91 : 5013834.2



**Council well number** : G41/0459
**Well name** : -
**Drilling company** : McNeill Drilling
**Drilling date** : 17/11/2015
**Drilling method** : Dual Rotary
**Locality** : -
**Total depth drilled** : 30.17m
**NZTM** : 1304356 : 5013954



# Appendix 2

## Geotago Ltd Site Plans and Test Pit Logs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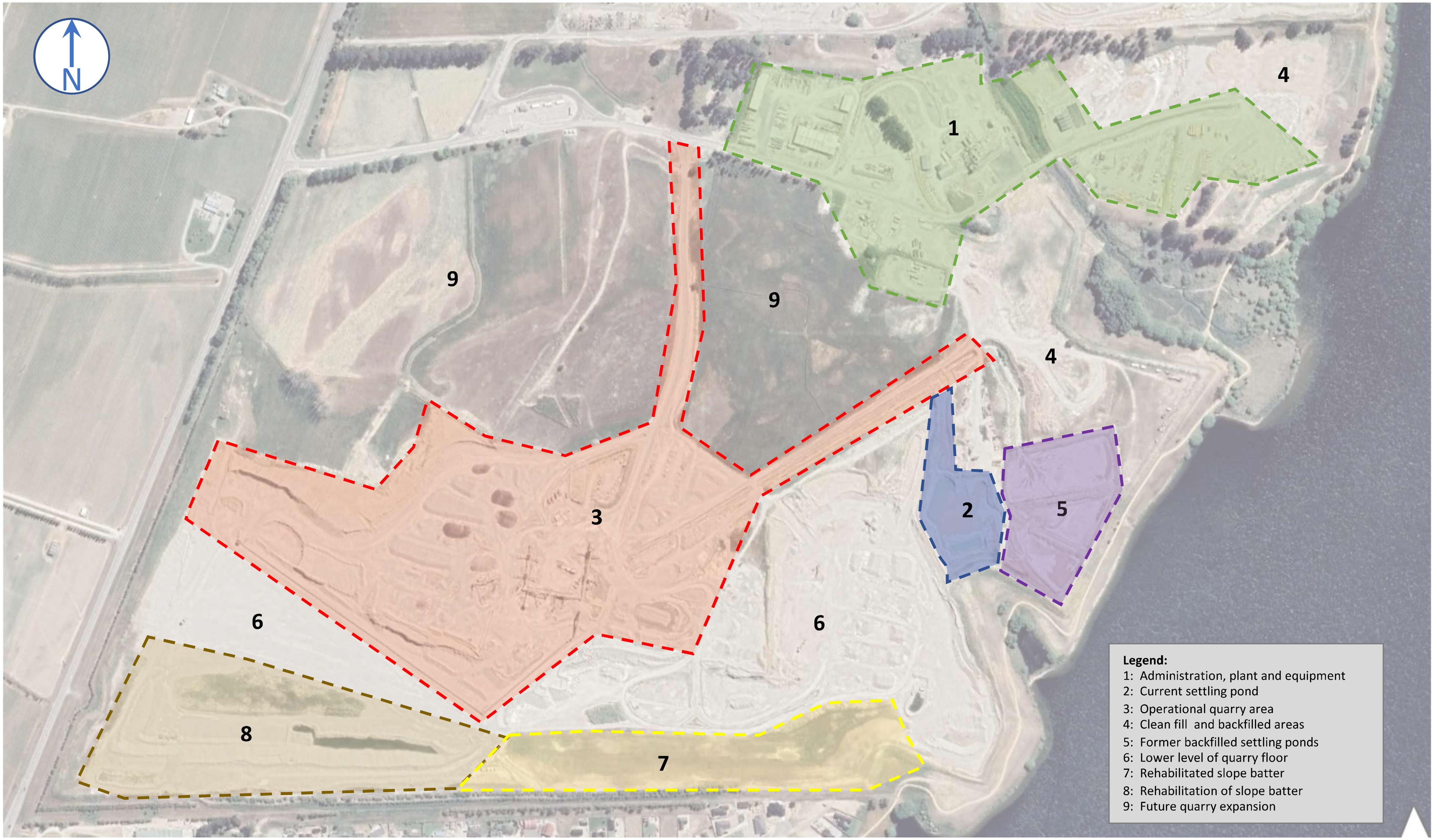
FULTON HOGAN

PARKBURN QUARRY, MOUNT PISA

SITE INVESTIGATION PLAN

| | |
|---|---|
| SCALE: 1:7000 APPROX<br>0m 280m | |
| REF:<br>21-022.1 DRW002 | DATE:<br>23 FEBRUARY 2022 |

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FULTON HOGAN

PARKBURN QUARRY, MOUNT PISA

SITE MODEL

SCALE: 1:7000 APPROX

0m 280m

REF: 21-022.1 DRW003

DATE: 23 FEBRUARY 2022

# Test Pit Log

| | | | |
|---|---|---|---|
| Project: | Parkburn Masterplan | Project Number: | GL21-022 |
| Site Location: | Parkburn Quarry, Mount Pisa | Client: | Fulton Hogan |

| | | |
|---|---|---|
| Test Pit Number: | TP101 | Sheet 1 of 1 |

| Depth (m) | Water Level | Geological Unit | Sample | Soil Rock Description | Legend | Depth |
|---|---|---|---|---|---|---|
| | | FILL | | Sand with gravels, light brown. Tightly packed, dry. Sand coarse grained, gravels fine grained subangular to subrounded (CRUSHER DUST) | | |
| 0.5 – 1.0 | ▽ (1.0) | OUTWASH DEPOSITS | | Sandy GRAVEL with cobbles and occasional small boulder, greyish brown. Loose to tightly packed, dry. Sand coarse grained, gravels well graded subangular to subrounded. Cobble subrounded. Strong water ingress from 1.0m depth. | | 0.5 – 1.0 |
| 1.5 – 3.5 | | | | End of pit due to water table. | | 1.5 – 3.0 |

| | |
|---|---|
| Date Excavated: 22 February 2022 | Equipment: 35T tracked excavator with 1200mm tooth bucket |
| Logged By: PF | Contractor: Fulton Hogan |

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Notes:

# Test Pit Log

| Project: | Parkburn Masterplan | Project Number: | GL21-022 |
|---|---|---|---|
| Site Location: | Parkburn Quarry, Mount Pisa | Client: | Fulton Hogan |

| Test Pit Number: | TP102 | Sheet 1 of 1 |
|---|---|---|

| Depth (m) | Water Level | Geological Unit | Sample | Soil Rock Description | Legend | Depth |
|---|---|---|---|---|---|---|
| | | TOPSOIL | | Sandy SILT, with organic content, roots and rootlets. | | |
| 0.5 | | FILL | | Sand with gravels, light brown. Tightly packed, dry. Sand coarse grained, gravels fine grained subangular to subrounded (CRUSHER DUST) | | 0.5 |
| 1.0<br>1.5 | ▽ | OUTWASH DEPOSITS | | Sandy GRAVEL with cobbles and occasional small boulder, greyish brown. Loose to tightly packed, dry. Sand coarse grained, gravels well graded subangular to subrounded. Cobble subrounded. Storng water ingress at 1.8m. | | 1.0<br>1.5 |
| 2.0<br>2.5<br>3.0<br>3.5 | | | | End of pit due to water table | | 2.0<br>2.5<br>3.0<br>3.5 |

| | |
|---|---|
| Date Excavated: 22 February 2022 | Equipment: 35T tracked excavator with 1200mm tooth bucket |
| Logged By: PF | Contractor: Fulton Hogan |

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Notes:

# Test Pit Log

| | | | |
|---|---|---|---|
| Project: | Parkburn Masterplan | Project Number: | GL21-022 |
| Site Location: | Parkburn Quarry, Mount Pisa | Client: | Fulton Hogan |

| | | |
|---|---|---|
| Test Pit Number: | TP103 | **Sheet 1 of 1** |

| Depth (m) | Water Level | Geological Unit | Sample | Soil Rock Description | Legend | Depth |
|---|---|---|---|---|---|---|
| | | FILL | | Sand with gravels, light brown. Tightly packed, dry. Sand coarse grained, gravels fine grained subangular to subrounded (CRUSHER DUST) | | |
| 0.5 | | OUTWASH DEPOSITS | | Sandy GRAVEL with cobbles and occasional small boulder, greyish brown. Loose to tightly packed, dry. Sand coarse grained, gravels well graded subangular to subrounded. Cobble subrounded. Water ingress at 1.6m. | | 0.5 |
| 1.0 | | | | | | 1.0 |
| 1.5 | ▽ | | | | | 1.5 |
| | | | | End of pit due to water table | | |
| 2.0 | | | | | | 2.0 |
| 2.5 | | | | | | 2.5 |
| 3.0 | | | | | | 3.0 |
| 3.5 | | | | | | 3.5 |

| | |
|---|---|
| Date Excavated: 22 February 2022 | Equipment: 35T tracked excavator with 1200mm tooth bucket |
| Logged By: PF | Contractor: Fulton Hogan |

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Notes:

# Test Pit Log

| | | | |
|---|---|---|---|
| Project: | Parkburn Masterplan | Project Number: | GL21-022 |
| Site Location: | Parkburn Quarry, Mount Pisa | Client: | Fulton Hogan |

| | | |
|---|---|---|
| Test Pit Number: | TP104 | **Sheet 1 of 1** |

| Depth (m) | Water Level | Geological Unit | Sample | Soil Rock Description | Legend | Depth |
|---|---|---|---|---|---|---|
| 0.0–0.3 | | TOPSOIL | | Sandy SILT, with organic content, roots and rootlets. | | |
| 0.3–0.8 (0.5) | | LOESS | | SILT, light brown. Tightly packed, dry. | | 0.5 |
| 0.8–2.2 (1.0, 1.5, 2.0) | | OUTWASH DEPOSITS | | Sandy GRAVEL with cobbles and occasional small boulder, greyish brown. Loose to tightly packed, dry. Sand coarse grained, gravels well graded subangular to subrounded. Cobble subrounded. | | 1.0, 1.5, 2.0 |
| 2.5, 3.0, 3.5 | | | | End of pit - Stable | | 2.5, 3.0, 3.5 |

| | |
|---|---|
| Date Excavated: 22 February 2022 | Equipment: 35T tracked excavator with 1200mm tooth bucket |
| Logged By: PF | Contractor: Fulton Hogan |

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Notes:

# Test Pit Log

| | | | |
|---|---|---|---|
| Project: | Parkburn Masterplan | Project Number: | GL21-022 |
| Site Location: | Parkburn Quarry, Mount Pisa | Client: | Fulton Hogan |

| | | |
|---|---|---|
| Test Pit Number: | TP105 | **Sheet 1 of 1** |

| Depth (m) | Water Level | Geological Unit | Sample | Soil Rock Description | Legend | Depth |
|---|---|---|---|---|---|---|
| 0.0–0.2 | | TOPSOIL | | Sandy SILT, with organic content, roots and rootlets. | | |
| 0.2–2.0 (0.5, 1.0, 1.5, 2.0) | | OUTWASH DEPOSITS | | Sandy GRAVEL with cobbles and occasional small boulder, greyish brown. Loose to tightly packed, dry. Sand coarse grained, gravels well graded subangular to subrounded. Cobble subrounded. | | 0.5, 1.0, 1.5, 2.0 |
| 2.0–3.5 (2.5, 3.0, 3.5) | | | | End of pit - stable | | 2.5, 3.0, 3.5 |

| | |
|---|---|
| Date Excavated: 22 February 2022 | Equipment: 35T tracked excavator with 1200mm tooth bucket |
| Logged By: PF | Contractor: Fulton Hogan |

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# Test Pit Log

| Project: | Parkburn Masterplan | Project Number: | GL21-022 |
|---|---|---|---|
| Site Location: | Parkburn Quarry, Mount Pisa | Client: | Fulton Hogan |

| Test Pit Number: | TP106 | Sheet 1 of 1 |
|---|---|---|

| Depth (m) | Water Level | Geological Unit | Sample | Soil Rock Description | Legend | Depth |
|---|---|---|---|---|---|---|
| | | TOPSOIL | | Sandy SILT, with organic content, roots and rootlets. | | |
| 0.5 – 2.0 | | OUTWASH DEPOSITS | | Sandy GRAVEL with cobbles and occasional small boulder, greyish brown. Loose to tightly packed, dry. Sand coarse grained, gravels well graded subangular to subrounded. Cobble subrounded. | | 0.5 – 2.0 |
| 2.5 – 3.5 | | | | End of pit - stable | | 2.5 – 3.5 |

| Date Excavated: 22 February 2022 | Equipment: 35T tracked excavator with 1200mm tooth bucket |
|---|---|
| Logged By: PF | Contractor: Fulton Hogan |

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Notes:

# Test Pit Log

| | | | |
|---|---|---|---|
| Project: | Parkburn Masterplan | Project Number: | GL21-022 |
| Site Location: | Parkburn Quarry, Mount Pisa | Client: | Fulton Hogan |

| | | |
|---|---|---|
| Test Pit Number: | TP107 | Sheet 1 of 1 |

| Depth (m) | Water Level | Geological Unit | Sample | Soil Rock Description | Legend | Depth |
|---|---|---|---|---|---|---|
| | | TOPSOIL | | Sandy SILT, with organic content, roots and rootlets. | | |
| 0.5 | | LOESS | | SILT, light brown. Tightly packed, dry. | | 0.5 |
| 1.0<br>1.5<br>2.0 | | OUTWASH DEPOSITS | | Sandy GRAVEL with cobbles and occasional small boulder, greyish brown. Loose to tightly packed, dry. Sand coarse grained, gravels well graded subangular to subrounded. Cobble subrounded. | | 1.0<br>1.5<br>2.0 |
| 2.5<br>3.0<br>3.5 | | | | End of pit - stable | | 2.5<br>3.0<br>3.5 |

| | |
|---|---|
| Date Excavated: 22 February 2022 | Equipment: 35T tracked excavator with 1200mm tooth bucket |
| Logged By: PF | Contractor: Fulton Hogan |

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Notes:

# Test Pit Log

| | | | |
|---|---|---|---|
| Project: | Parkburn Masterplan | Project Number: | GL21-022 |
| Site Location: | Parkburn Quarry, Mount Pisa | Client: | Fulton Hogan |

| | | |
|---|---|---|
| Test Pit Number: | TP108 | Sheet 1 of 1 |

| Depth (m) | Water Level | Geological Unit | Sample | Soil Rock Description | Legend | Depth |
|---|---|---|---|---|---|---|
| 0.0–0.7 | | UNCONTROLLED FILL | | Sandy silty GRAVELS with cobbles and occasional extraneous materials (wood, plastic, concrete). Grey brown. Tightly packed, dry. | | |
| 0.7–0.9 | | | | Sandy silt with roots, rootlets and decaying plant matter | | |
| 0.9–2.0 | | | | Sandy GRAVEL, light browish grey. Very loosely packed, dry. Gravels 5-8mm in diameter. ('PEAS') | | |
| 2.0 | | | | End of pit - collapsing in due to pea gravel | | |

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5

| | |
|---|---|
| Date Excavated: 22 February 2022 | Equipment: 35T tracked excavator with 1200mm tooth bucket |
| Logged By: PF | Contractor: Fulton Hogan |

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Notes:

# Test Pit Log

| Project: | Parkburn Masterplan | Project Number: | GL21-022 |
|---|---|---|---|
| Site Location: | Parkburn Quarry, Mount Pisa | Client: | Fulton Hogan |

| Test Pit Number: | TP109 | Sheet 1 of 1 |
|---|---|---|

| Depth (m) | Water Level | Geological Unit | Sample | Soil Rock Description | Legend | Depth |
|---|---|---|---|---|---|---|
| 0.0–0.3 | | TOPSOIL | | Sandy SILT, with organic content, roots and rootlets. | | |
| 0.3–2.0 | | UNCONTROLLED FILL | | Sandy GRAVEL, light browish grey. Very loosely packed, dry. Gravels 5-8mm in diameter. ('PEAS') | | |
| 2.0 | | | | End of pit - collapsing in the pea gravel | | |

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5

| | |
|---|---|
| Date Excavated: 22 February 2022 | Equipment: 35T tracked excavator with 1200mm tooth bucket |
| Logged By: PF | Contractor: Fulton Hogan |


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Notes:

# Test Pit Log

| | | | |
|---|---|---|---|
| Project: | Parkburn Masterplan | Project Number: | GL21-022 |
| Site Location: | Parkburn Quarry, Mount Pisa | Client: | Fulton Hogan |

| | | |
|---|---|---|
| Test Pit Number: | TP110 | **Sheet 1 of 1** |

| Depth (m) | Water Level | Geological Unit | Sample | Soil Rock Description | Legend | Depth |
|---|---|---|---|---|---|---|
| | | TOPSOIL | | SILT, light brown. Tightly packed, dry. | | |
| 0.5 | | UNCONTROLLED FILL | | Sand with gravels, light brown. Tightly packed, dry. Sand coarse grained, gravels fine grained subangular to subrounded (CRUSHER DUST) | | 0.5 |
| 1.0 / 1.5 / 2.0 | | UNCONTROLLED FILL | | Sandy GRAVEL, light browish grey. Very loosely packed, dry. Gravels 5-8mm in diameter. ('PEAS') | | 1.0 / 1.5 / 2.0 |
| 2.5 / 3.0 / 3.5 | | | | End of pit - collapsing in pea gravels | | 2.5 / 3.0 / 3.5 |

| | |
|---|---|
| Date Excavated: 22 February 2022 | Equipment: 35T tracked excavator with 1200mm tooth bucket |
| Logged By: PF | Contractor: Fulton Hogan |

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Notes:

# Test Pit Log

| Project: | Parkburn Masterplan | Project Number: | GL21-022 |
|---|---|---|---|
| Site Location: | Parkburn Quarry, Mount Pisa | Client: | Fulton Hogan |
| Test Pit Number: | TP111 | | Sheet 1 of 1 |

| Depth (m) | Water Level | Geological Unit | Sample | Soil Rock Description | Legend | Depth |
|---|---|---|---|---|---|---|
| 0.0–0.3 | | TOPSOIL | | Sandy SILT, with organic content, roots and rootlets. | | |
| 0.5 | | LOESS | | SILT, light brown. Tightly packed, dry. | | 0.5 |
| 1.0<br>1.5<br>2.0 | | OUTWASH DEPOSITS | | Sandy GRAVEL with cobbles and occasional small boulder, greyish brown. Loose to tightly packed, dry. Sand coarse grained, gravels well graded subangular to subrounded. Cobble subrounded. | | 1.0<br>1.5<br>2.0 |
| 2.5<br>3.0<br>3.5 | | | | End of pit - stable | | 2.5<br>3.0<br>3.5 |

| | |
|---|---|
| Date Excavated: 22 February 2022 | Equipment: 35T tracked excavator with 1200mm tooth bucket |
| Logged By: PF | Contractor: Fulton Hogan |

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Notes:

# Test Pit Log

| | | | |
|---|---|---|---|
| Project: | Parkburn Masterplan | Project Number: | GL21-022 |
| Site Location: | Parkburn Quarry, Mount Pisa | Client: | Fulton Hogan |

| | | |
|---|---|---|
| Test Pit Number: | TP112 | Sheet 1 of 1 |

| Depth (m) | Water Level | Geological Unit | Sample | Soil Rock Description | Legend | Depth |
|---|---|---|---|---|---|---|
| | | TOPSOIL | | Sandy SILT, with organic content, roots and rootlets. | | |
| 0.5, 1.0 | | LOESS | | SILT, light brown. Tightly packed, dry. | | 0.5, 1.0 |
| 1.5, 2.0 | | OUTWASH DEPOSITS | | Sandy GRAVEL with cobbles and occasional small boulder, greyish brown. Loose to tightly packed, dry. Sand coarse grained, gravels well graded subangular to subrounded. Cobble subrounded. | | 1.5, 2.0 |
| 2.5, 3.0, 3.5 | | | | End of pit - stable | | 2.5, 3.0, 3.5 |

| | |
|---|---|
| Date Excavated: 22 February 2022 | Equipment: 35T tracked excavator with 1200mm tooth bucket |
| Logged By: PF | Contractor: Fulton Hogan |

Geotago Ltd
Arrow Junction
Queenstown 9371
New Zealand

T: +64 272 699 736
E: pete@geotago.nz
W: www.geotago.nz

geotago
Engineering Geology & Geotechnics

Notes:

# Appendix 3

## Crushed Pea Gravel Compaction Trial Report (2025)

3 November 2025

Fulton Hogan Land Development Ltd
C/- Tim James
Via email: tjdjames.nz@gmail.com
CC: graeme.causer@fultonhogan.com

# Re: Crushed Pea Gravel Compaction Trial, Parkburn Quarry

**Our Reference: 24019_1**

## 1 Introduction and Scope of Report

JKCM Ltd, trading as Insight Engineering (IE), was requested to arrange a field compaction trial of crushed pea gravels sourced from Parkburn Quarry, using a hardfill plateau density test procedure.

This letter presents the methods used to complete the trial and results including relative compaction and water content, as well as estimations of volumetric compression of the crushed pea gravel. We also present our opinion on the use of this material as a bulk fill material, based on the trial and test results.

## 2 Methodology

### 2.1 Location and Operators

The field compaction trial was carried out at the Fulton Hogan Miners Road Quarry in Christchurch. Machinery and operators used in the trial were supplied by Fulton Hogan, and field density testing was carried out by Fulton Hogan Canterbury Laboratory.

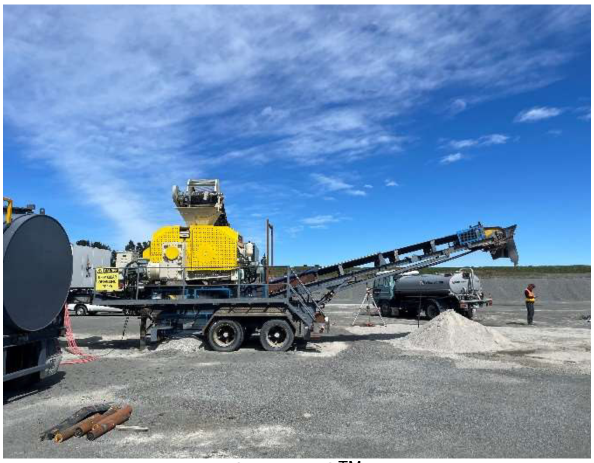
### 2.2 Hardfill Materials

The materials used in the trial were comprised of pea gravel which was transported from Parkburn Quarry. The pea gravel was crushed using a high pressure grinding roll crusher (HRC™800), operating at a rate of approximately 90 ton/hour. The crushed gravel product is represented by the Particle Size Distribution plot presented as Appendix 1 to this letter. Based on this plot, the crushed product can be described as sandy, silty, fine to medium GRAVEL.

The sand and gravel fragments were observed to be typically sub-rounded to angular following crushing.

The crushed gravel was generally dry to moist when taken from the stockpile, and water had to be added prior to compacting. Water was sprayed from a water truck for around 5-10 seconds for each trial hardfill pad.

### Figure 1: Materials Photographs



Photograph 1: View of the HRC™800 crushing plant and showing a small stockpile of crushed gravel.

Photograph 2: View of a sample of crushed gravel

## 2.3 Base Preparation

The ground surface within the quarry where the trial pads were to be formed was very hard. Therefore, to simulate less favourable base conditions, the subgrade was excavated to a depth of approximately 0.4 m. Crushed pea gravel was used to backfill the excavation. The crushed gravel was lightly compacted using 4 single passes of a 15.5T vibrating drum roller.

This resulted in a relatively firm base with a dry density of 1.64 t/m$^3$.

### Figure 2: Base Preparation Photographs

Photograph 3: Over-excavation of the base.

Photograph 4: View of the prepared base prior to formation of the trial hardfill pads.

## 2.4 Trial Hardfill Pads

The trial hardfill pads were constructed on top of the prepared base. Due to the limited quantity of crushed gravel available, only one hardfill pad could be constructed and tested at a time. The width and length of each pad was approximately 3 m and 5 m respectively.

Trial pad 1 was formed using a 0.4 m thick loose layer of crushed gravel.

Trial pad 2 was formed using a 0.5 m thick loose layer of crushed gravel.

Trial pad 3 was formed using a 0.6 m thick loose layer of crushed gravel.

## 2.5 Compaction and Testing

Each pad was prepared, compacted and testing following the same methodology.

Compaction was achieved with a 15.5T vibrating drum roller. Initial compaction involved four single passes of the roller (i.e. a pass was recorded as one way, not back and forth). Nuclear Density testing (NDM) on backscatter mode was carried out after the initial four single passes, and then after every two single passes until a plateau was achieved for each hardfill pad. Static rolling was then carried out with testing completed at the same intervals until a plateau was again achieved.

Additional NDM testing using the Direct Transmission method was carried out upon completion of compaction on each trial pad to measure the dry density at depths of 250mm, 150mm and 100mm below the surface of the hardfill pads.

Supplementary testing to correlate the *insitu* CBR was undertaken using an Impact Tester (Clegg Hammer).

Additionally, upon completion of the plateau testing on the third trial pad (600 mm thick loose layer), we completed two Light Weight Deflectometer tests (LWD) to estimate the stiffness of the compacted hardfill layer.

**Figure 3: Compaction and Testing**

Photograph 5: Trial pad 1 prior to compaction.

Photograph 6: Testing of Trial pad 3.

# 3 Results

The plateau test results are presented as Appendix 2 of this letter. The laboratory Standard Compaction test result is presented as Appendix 3.

## 3.1 Dry Density

The laboratory compaction test indicates a maximum dry density (MDD) of 2.06 t/m$^3$ with an optimal water content of 6%.

The plateau tests give the field maximum dry density (field MDD) of the crushed gravel as used in the trial. The field MDD is taken as 2.07 t/m$^3$ from the Plateau Test #2 result, which relates to 18 single passes of the compactor (or 9 passes back and forth). The water content of the hardfill pad was recorded as 3.7% during the trial.

Dry density results from the direct transmission tests for all three trial pads were equal to or exceeding 95% of the laboratory MDD, which equates to 1.96 t/m$^3$.

## 3.2 CIV and CBR

The impact value (CIV) was generally returned as 10 to 15 following 10 single passes of the roller. This value correlates[1] to an estimated CBR of 7%.

## 3.3 Light Weight Deflectometer

The dynamic modulus is indicated by the LWD, and for the trial hardfill pad the modulus values were returned as approximately 24 and 26 $MN/m^2$.

## 3.4 Compression of the hardfill pads

Compression of the hardfill following compaction was estimated by recording pre-compaction elevations of each trial pad using a dumpy level. The elevations were recorded again once the compaction and plateau testing of each trial pad was completed.

Trial pad 1 was compressed by an estimated 100 to 150mm.

Trial pad 2 was compressed by an estimated 140 to 200 mm.

Trial pad 3 was compressed by an estimated 150 mm.

It should be noted that these quantities are estimates only and lateral displacement may have also contributed to the overall vertical compression. It was not possible to estimate lateral displacement with any accuracy during the trials.

# 4 Discussion on Results

The field MDD obtained from the plateau tests is greater than the laboratory MDD. This is likely due to the field machinery being able to provide a higher compactive effort than the standardised laboratory equipment. However, the difference between the field and laboratory MDD is not considered to be significant in this case, and therefore minimum target dry densities of 95% or 98% (subject to fill volume, thickness and final land use) should be achievable in the production of bulk fill using crushed pea gravel.

From the trials, we consider that the optimal case for compaction is likely to be achieved using loose layers of 500 mm (as indicated by plateau test #2), providing that similar sized compaction machinery is used. Our reasoning for this conclusion is based on the number of passes to achieve the field MDD and water content, as well as the results of the direct transmission tests. Note that the plateau test indicated 18 single passes which can also be taken as 9 passes back and forth.

It is our opinion that adequate compaction is also likely to be achievable using loose layers of 600 mm thick. However, more water and suitably heavy (greater than 15T) compaction machinery moving at low speeds is likely to be required to achieve consistent results, which may be less efficient.

It is interesting to note that the results from plateau test # 1 (for the 400 mm thick loose layer) are less optimal than for #2. We anticipate that this is likely due to the prepared base, which had stiffened up during plateau test # 1, therefore providing slightly more favourable conditions for plateau test # 2. Therefore, for future areas where the bulk fill is placed over less favourable subgrade, thinner loose layers may be more appropriate to ensure adequate compaction is achieved throughout the fill layer.

[1] $CBR = 0.07 \times (CIV)^2$ and $CBR = (0.24 \times CIV+1)^2$

Volumetric compression of the hardfill pads were typically indicated to be around 25% but potentially up to 40%. However, as indicated above we are unclear how much lateral displacement has contributed to the compression.

The Impact Test results indicated that a CBR of 7 is likely to be achievable where the appropriate degree of compaction is carried out. The laboratory soaked CBR tests results indicate significantly higher values, however in this case we consider it is prudent to refer to the field test results over the laboratory results.

The LWD indicated dynamic modulii of 24 and 26 MN/$m^2$ for the third hardfill pad (600 mm loose layer thickness). This result indicates a moderately stiff subgrade is likely to be achieved, which is generally in keeping with our assessment of the field CBR from the Impact Test and the achieved dry density values.

# 5 Other Considerations

The crushed gravel is likely to be produced as a uniform, consistent product which allows for more efficient on site quality controls during bulk earthworks. Gravel processed at Parkburn Quarry is likely to be drier than materials used for the trials, and addition of water must be allowed for, both for compaction and for dust control.

Wet weather conditions are unlikely to significantly disrupt fill placement activities using crushed pea gravel.

Larger compaction machinery could be more efficient, requiring fewer passes and potentially allowing for thicker loose layers. Further trials to refine the earthfilling specification could be carried out at Parkburn in the future with actual machinery proposed to be used, as was done for Pembroke Heights.

In addition to the volumetric compression, settlement of fill due to self weight is a potentially complex issue that should require further assessment, particularly given the large volumes of fill proposed to be greater than 6 m thick. This is estimated to potentially range between 0.5 and 2 % of the total fill thickness, subject to the quality of the fill and time taken to construct. Potential options to mitigate effects of fill settlement may involve temporarily over-filling fill areas to induce settlement, prior to constructing pavements, services and final surfaces etc. It should also be noted that the trial was carried out at a water content that is less than optimal, based on the laboratory compaction test. This may have an impact on the overall air voids after compaction, which may also contribute to settlement.

The pea gravel is expected to be relatively durable and chemical or other weathering in the long term, that could potentially compromise the material as a fill product is not anticipated to be an issue. However, for confirmation, we recommend that mineralogical and aggregate testing is carried out on a representative sample of pea gravel.

Landscaping areas that are proposed to be vegetated with trees etc., may require a different approach to facilitate proper growth and allow the survival of plants, as tightly compacted crushed gravel may not provide optimal growing conditions.

# 6 Limitations

i. We have prepared this report in accordance with the brief as provided. This report has been prepared for the use of our client, Fulton Hogan Land Development Ltd, their professional advisers and the relevant Territorial Authorities in relation to the specified project brief described in this report. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity.

ii. The recommendations in this report are based on the ground and material conditions indicated from published sources, inspections and tests described in this report based on accepted normal methods of site investigations. Only a limited amount of information has been collected to meet the specific financial and technical requirements of the Client's brief and this report does not purport to completely describe all the site and material characteristics and properties. The nature and continuity of the ground between test locations has been inferred using experience and judgement and it must be appreciated that actual conditions could vary from the assumed model.

iii. Subsurface conditions relevant to construction works should be assessed by contractors who can make their own interpretation of the factual data provided. They should perform any additional tests as necessary for their own purposes.

iv. This Limitation should be read in conjunction with the IPENZ/ACENZ Standard Terms of Engagement.

v. This report is not to be reproduced either wholly or in part without our prior written permission.

## Report prepared by

Jana Kruyshaar, CMEngNZ, MIPENZ
Principal Geotechnical Engineer

Appendix 1: Laboratory Particle Size Distribution Test Result

Appendix 2: Plateau Testing Results

Appendix 3: Laboratory Standard Compaction Test Result

# Appendix 1

## Laboratory Particle Size Distribution Test Result

**Work Order ID:** CAN25W7644

**Canterbury Laboratory**
24 Miners Road, Templeton, Christchurch
PO Box 16-064, Christchurch 8441
**Telephone:** +64 3 349 9142
**www.fultonhogan.com**
0800 LABORATORY

**Report No: MAT:CAN25S-21825**
**Issue No: 1**

# Material Test Report

**Client:**
Cant Land Resource
Corner Main South & Halswell Junction Rd
Hornby

Christchurch 8042
NZ

**Project:** 251706 Parkburn Quarry-Cant Land Resource SWO INT

The tests reported herein (unless otherwise indicated) have been performed in accordance with the laboratory's scope of accreditation. Samples are tested as received, in natural condition, unless stated otherwise in the comments. This report may only be reproduced in full.

The results in this report relate only to the items / samples that were tested

Approved Signatory: Liam Brennan
(Laboratory Operations Manager)
IANZ Accreditation No:200
Date of Issue: 29/10/2025

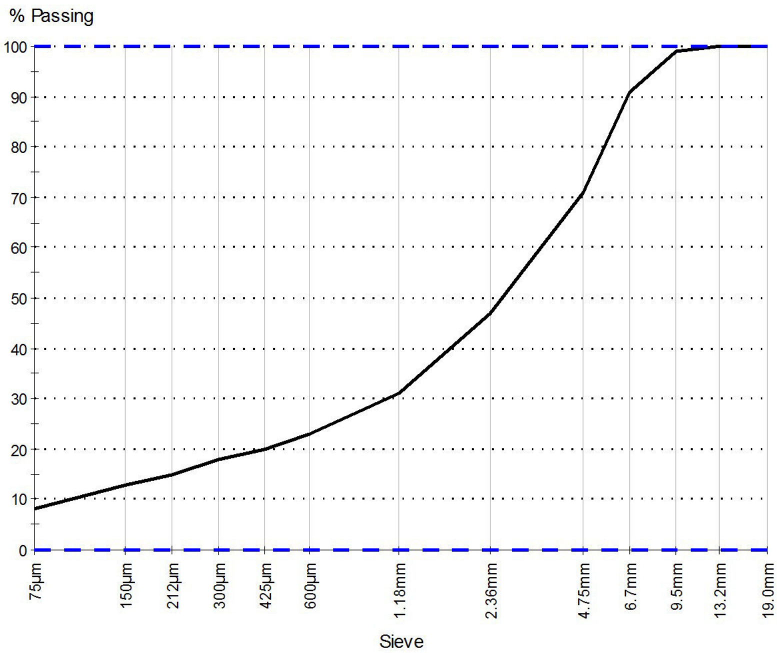
## Sample Details

**Sample ID:** CAN25S-21825
**Client Sample ID:** QA Testing
**Material:** Crusher Dust
**Sample Source:** Miscellaneous Material Source
**Site/Sampled From:** Miners Rd West ,ex Parkburn Quarry Trial Pad
**Date Sampled:** 22/10/2025
**Specification:** No Specification
**Sampled By:** Liam Brennan
**Sampling Method:** NZS 4407:2015 2.4.6.3.2 (SP/WG/MACHINE)
**Technician:** Amanda Ryan
**Sampling Endorsed?:** Yes

## Other Test Results

| Description | Method | Result | Limits |
|---|---|---|---|

## Particle Size Distribution




**Method:** NZS 4407:2015 Test 3.8.2
**Drying By:** Oven
**Date Tested:** 27/10/2025
**Tested By:** Amanda Ryan

| Sieve Size | % Passing | Limits |
|---|---|---|
| 19.0mm | 100 | 0 – 100 |
| 13.2mm | 100 | 0 – 100 |
| 9.5mm | 99 | 0 – 100 |
| 6.7mm | 91 | 0 – 100 |
| 4.75mm | 71 | 0 – 100 |
| 2.36mm | 47 | 0 – 100 |
| 1.18mm | 31 | 0 – 100 |
| 600µm | 23 | 0 – 100 |
| 425µm | 20 | 0 – 100 |
| 300µm | 18 | 0 – 100 |
| 212µm | 15 | 0 – 100 |
| 150µm | 13 | 0 – 100 |
| 75µm | 8 | 0 – 100 |

## Comments

N/A

# Appendix 2

## Plateau Test Result

Canterbury Laboratory
24 Miners Road, Templeton
PO Box 16 064, Hornby
Christchurch
Phone (03) 349 9142
Email:
cantlab@fultonhogan.com
0800 LABORATORY

# Plateau Test Report

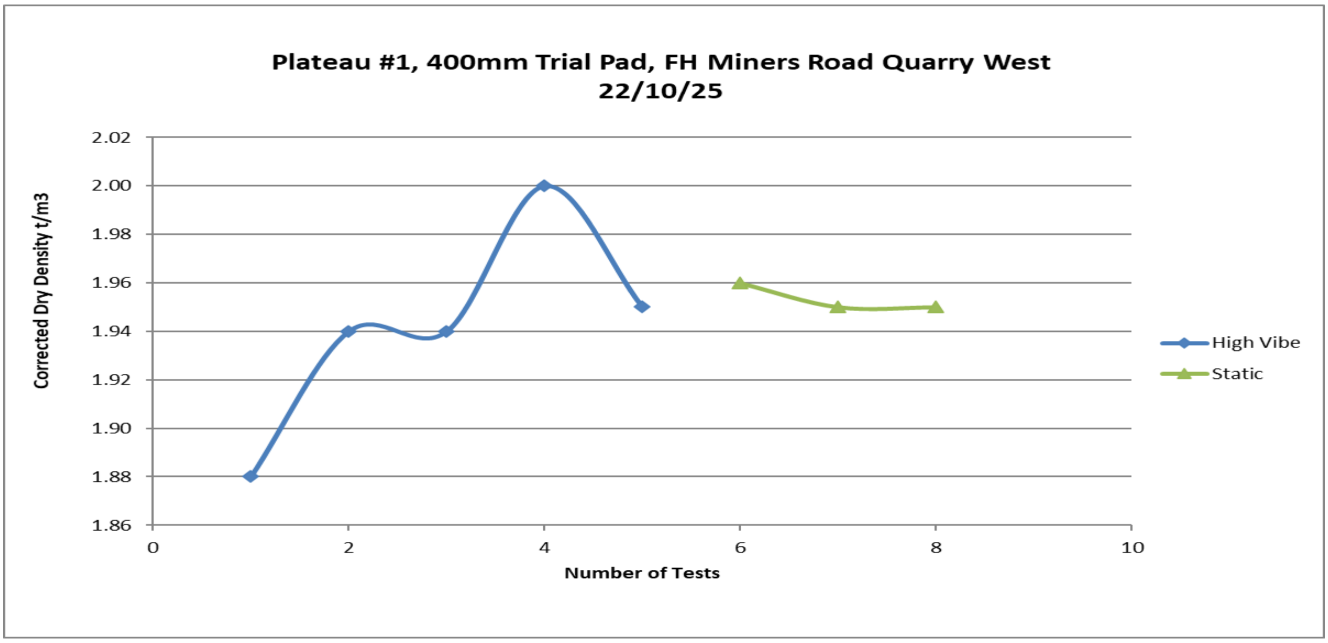
| | | | |
|---|---|---|---|
| **Client:** | Cant Land Resource | **Report No:** | **CAN25W7541** |
| **Project:** | **2500173W3017512** | **Issue No:** | **1** |

| **Testing Details** | |
|---|---|
| Site Tested: | 400mm Trial Pad, FH Miners Road Quarry West |
| Tested by: | Liam Brennan |
| Date: | 22/10/25 |
| Time Tested: | 09:00am |
| Material: | Crushed Parkburn Quarry Pea Gravel |
| Field Method: | NZS 4407:2015 Test 4.3 |
| Lab Method: | NZS 4407:2015 Test 3.1 |

## Plateau Test #1

| Test Results | | | | | |
|---|---|---|---|---|---|
| **Test No.** | **Passes** | **Roller Used** | **Compaction Stage** | **Dry Density ($t/m^3$)** | **Average Moisture Content (%)** |
| 1 | **4** | **15.5T Construction Roller** | **High Vibe** | 1.88 | **3.1** |
| 2 | **2** | | | 1.94 | |
| 3 | **2** | | | 1.94 | |
| 4 | **2** | | | **2.00** | |
| 5 | **2** | | | 1.95 | |
| 6 | **2** | | **Static** | 1.96 | |
| 7 | **2** | | | 1.96 | |
| 8 | **2** | | | 1.95 | |

Please note: One pass is in a single direction over the test spot.



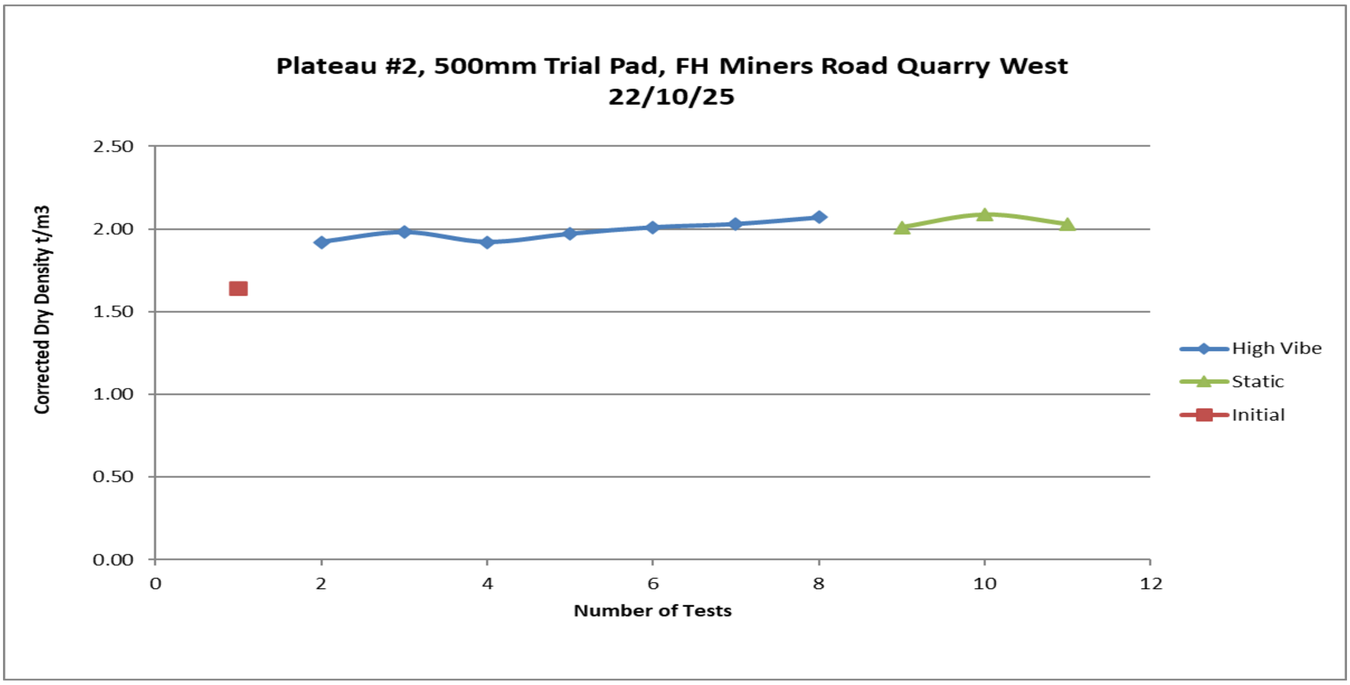
**Canterbury Laboratory**
24 Miners Road, Templeton
PO Box 16 064, Hornby
Christchurch
Phone (03) 349 9142
Email:
cantlab@fultonhogan.com
0800 LABORATORY

| **Testing Details** | |
|---|---|
| Site Tested: | 500mm Trial Pad, FH Miners Road Quarry West |
| Tested by: | Liam Brennan |
| Date: | 22/10/25 |
| Time Tested: | 10:00am |
| Material: | Crushed Parkburn Quarry Pea Gravel |
| Field Method: | NZS 4407:2015 Test 4.3 |
| Lab Method: | NZS 4407:2015 Test 3.1 |

**Plateau Test #2**

| Test No. | Passes | Roller Used | Compaction Stage | Dry Density ($t/m^3$) | Average Moisture Content (%) |
|---|---|---|---|---|---|
| 1 | **0** | **N/A** | **Initial - Laid** | 1.64 | **3.7** |
| 2 | **4** | **15.5T Construction Roller** | **High Vibe** | 1.92 | |
| 3 | **2** | | | 1.98 | |
| 4 | **2** | | | 1.92 | |
| 5 | **2** | | | 1.97 | |
| 6 | **2** | | | 2.02 | |
| 7 | **2** | | | 2.03 | |
| 8 | **2** | | | 2.07 | |
| 9 | **2** | | **Static** | 2.01 | |
| 10 | **2** | | | **2.08** | |
| 11 | **2** | | | 2.04 | |

Please note: One pass is in a single direction over the test spot.



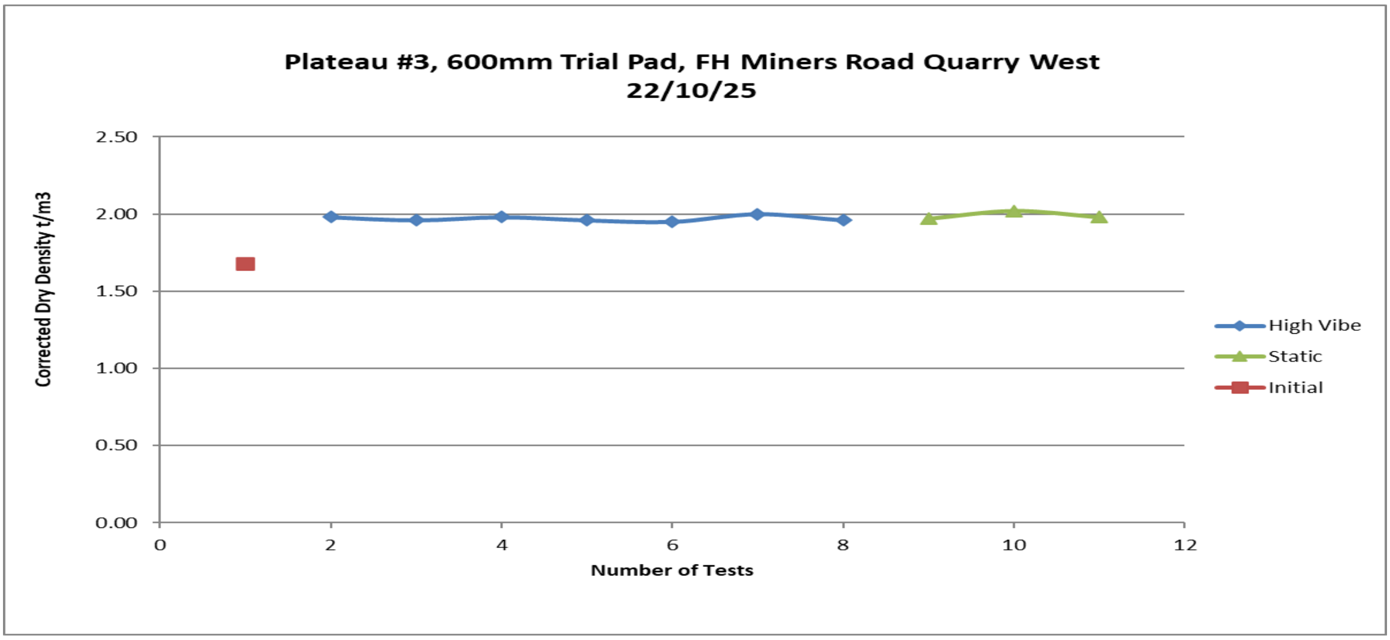
**Canterbury Laboratory**
24 Miners Road, Templeton
PO Box 16 064, Hornby
Christchurch
Phone (03) 349 9142
Email:
cantlab@fultonhogan.com
0800 LABORATORY

| **Testing Details** | |
|---|---|
| Site Tested: | 600mm Trial Pad, FH Miners Road Quarry West |
| Tested by: | Liam Brennan |
| Date: | 22/10/25 |
| Time Tested: | 10:45am |
| Material: | Crushed Parkburn Quarry Pea Gravel |
| Field Method: | NZS 4407:2015 Test 4.3 |
| Lab Method: | NZS 4407:2015 Test 3.1 |

## Plateau Test #3

| Test No. | Passes | Roller Used | Compaction Stage | Dry Density ($t/m^3$) | Average Moisture Content (%) |
|---|---|---|---|---|---|
| 1 | **0** | **N/A** | **Initial - Laid** | 1.68 | **5.3** |
| 2 | **4** | **15.5T Construction Roller** | **High Vibe** | 1.98 | |
| 3 | **2** | | | 1.96 | |
| 4 | **2** | | | 1.98 | |
| 5 | **2** | | | 1.96 | |
| 6 | **2** | | | 1.95 | |
| 7 | **2** | | | 2.00 | |
| 8 | **2** | | | 1.96 | |
| 9 | **2** | | **Static** | 1.97 | |
| 10 | **2** | | | **2.01** | |
| 11 | **2** | | | 1.98 | |

Please note: One pass is in a single direction over the test spot.



**Canterbury Laboratory**
24 Miners Road, Templeton
PO Box 16 064, Hornby
Christchurch
Phone (03) 349 9142
Email:
cantlab@fultonhogan.com
0800 LABORATORY

# Direct Transmission NDM Test Results:

Test method used: NZS 4407:2015 Test 4.2

| Test Results at the conclusion of Plateau Test – 400mm Trial Pad | | | | |
|---|---|---|---|---|
| **Test** | **Probe Depth (mm)** | **Dry Density (kg/m³)** | **Wet Density (kg/m³)** | **Moisture Content (%)** |
| **1** | **250** | 2.016 | 2.089 | 3.6 |
| **2** | **150** | 2.051 | 2.116 | 3.2 |
| **3** | **100** | 2.007 | 2.089 | 4.1 |
| **4** | **0** | 1.939 | 2.006 | 3.4 |

| Test Results at the conclusion of 8th High Vibe pass in the Plateau Test – 500mm Trial Pad | | | | |
|---|---|---|---|---|
| **Test** | **Probe Depth (mm)** | **Dry Density (kg/m³)** | **Wet Density (kg/m³)** | **Moisture Content (%)** |
| **1** | **250** | 1.988 | 2.047 | 2.7 |
| **2** | **150** | 1.961 | 2.028 | 3.4 |

| Test Results at the conclusion of 10th High Vibe pass in the Plateau Test – 500mm Trial Pad | | | | |
|---|---|---|---|---|
| **Test** | **Probe Depth (mm)** | **Dry Density (kg/m³)** | **Wet Density (kg/m³)** | **Moisture Content (%)** |
| **1** | **200** | 1.982 | 2.055 | 3.7 |
| **2** | **150** | 2.002 | 2.068 | 3.3 |

| Test Results at the conclusion of 2nd Static pass in the Plateau Test – 500mm Trial Pad | | | | |
|---|---|---|---|---|
| **Test** | **Probe Depth (mm)** | **Dry Density (kg/m³)** | **Wet Density (kg/m³)** | **Moisture Content (%)** |
| **1** | **250** | 2.025 | 2.097 | 3.6 |
| **2** | **150** | 2.026 | 2.098 | 3.6 |
| **3** | **100** | 2.006 | 2.076 | 3.5 |

| Test Results at the conclusion of Plateau Test – 600mm Trial Pad | | | | |
|---|---|---|---|---|
| **Test** | **Probe Depth (mm)** | **Dry Density (kg/m³)** | **Wet Density (kg/m³)** | **Moisture Content (%)** |
| **1** | **250** | 1.999 | 2.108 | 5.5 |
| **2** | **150** | 2.009 | 2.113 | 5.2 |
| **3** | **100** | 2.026 | 2.129 | 5.0 |
| **4** | **50** | 2.051 | 2.153 | 5.0 |

**Canterbury Laboratory**
24 Miners Road, Templeton
PO Box 16 064, Hornby
Christchurch
Phone (03) 349 9142
Email:
cantlab@fultonhogan.com
0800 LABORATORY

## Site Tested:

| | | |
|---|---|---|
| Report Checked By: | Liam Brennan | 23/10/2025 |
| Report Issued By: | Liam Brennan | 23/10/2025 |

# Appendix 3

## Laboratory Standard Compaction Test Result

**Work Order ID:** CAN25W7644

**Canterbury Laboratory**
24 Miners Road, Templeton, Christchurch
PO Box 16-064, Christchurch 8441
**Telephone:** +64 3 349 9142
**www.fultonhogan.com**
0800 LABORATORY

**Report No: MDD:CAN25S-21826**
**Issue No: 1**

# Maximum Dry Density Report

**Client:**
Cant Land Resource
Corner Main South & Halswell Junction Rd
Hornby

Christchurch 8042
NZ

**Project:** 251706 Parkburn Quarry-Cant Land Resource SWO INT

The tests reported herein (unless otherwise indicated) have been performed in accordance with the laboratory's scope of accreditation. Samples are tested as received, in natural condition, unless stated otherwise in the comments. This report may only be reproduced in full.

The results in this report relate only to the items / samples that were tested

Approved Signatory: Liam Brennan
(Laboratory Operations Manager)
IANZ Accreditation No:200
Date of Issue: 29/10/2025

## Sample Details

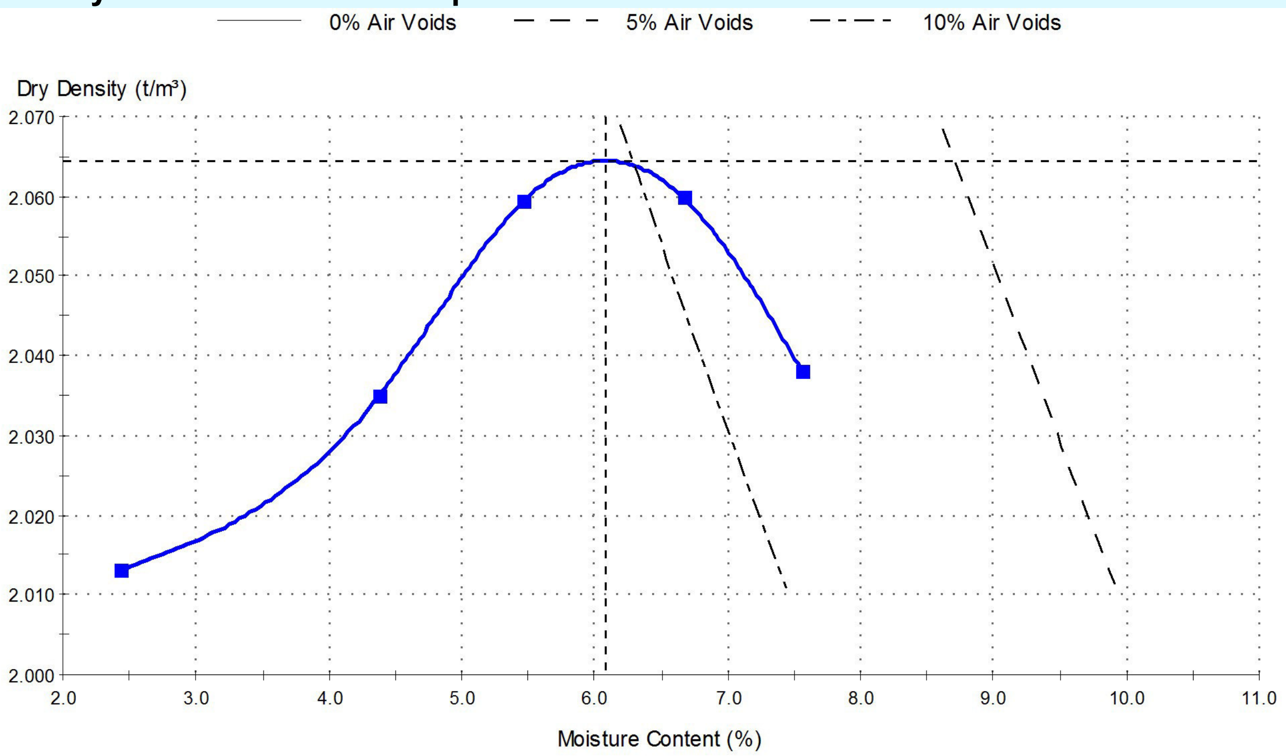
| | | | |
|---|---|---|---|
| **Sample ID:** | CAN25S-21826 | **Client Sample ID:** | QA Testing |
| **Material:** | Crusher Dust | **Sample Source:** | Miscellaneous Material Source |
| **Site/Sampled From:** | Miners Rd West ,ex Parkburn Quarry Trial Pad | **Date Sampled:** | 22/10/2025 |
| **Specification:** | Standard Compaction Test | **Sampled By:** | Liam Brennan |
| **Sampling Method:** | NZS 4407:2015 2.4.6.3.2 (SP/WG/MACHINE) | **Date Tested:** | 27/10/2025 |
| **Tested By:** | Amanda Ryan | **Sampling Endorsed?:** | Yes |

## Test Results

NZS 4402:1986 Test 4.1.1 - 1986

| | | | |
|---|---|---|---|
| **Maximum Dry Density (t/m³):** | **2.06** | **Optimum Moisture Content (%):** | **6.0** |
| **Solid Density (t/m³):** | 2.680 assumed | **Oversize Sieve (mm):** | 19.0 |
| **Oversize Material (%):** | 0 | **Sample History:** | Natural |

## Dry Density - Moisture Relationship




## Comments

# Appendix 4

## Crushed Pea Gravel Compaction Trial Test Results (2026)

**Work Order ID:** CTS26W0576

**Central Testing Services**
18 Ngapara Street, Alexandra 9340, Central Otago
P.O Box 397, Alexandra, Central Otago
**Telephone:** +64 3 4487644
**www.centraltesting.co.nz**

**Report No: MAT:CTS26S-00605**
**Issue No: 1**

# Material Test Report

**Client:**
Central Testing Cash Sales
PO Box 397

Alexandra 9340
NZ

**Project:** Insight Engineering

The tests reported herein (unless otherwise indicated) have been performed in accordance with the laboratory's scope of accreditation. Samples are tested as received, in natural condition, unless stated otherwise in the comments. This report may only be reproduced in full.

The results in this report relate only to the items / samples that were tested

Approved Signatory: Carl Julius
(Laboratory Technician)
IANZ Accreditation No:434
Date of Issue: 17/03/2026

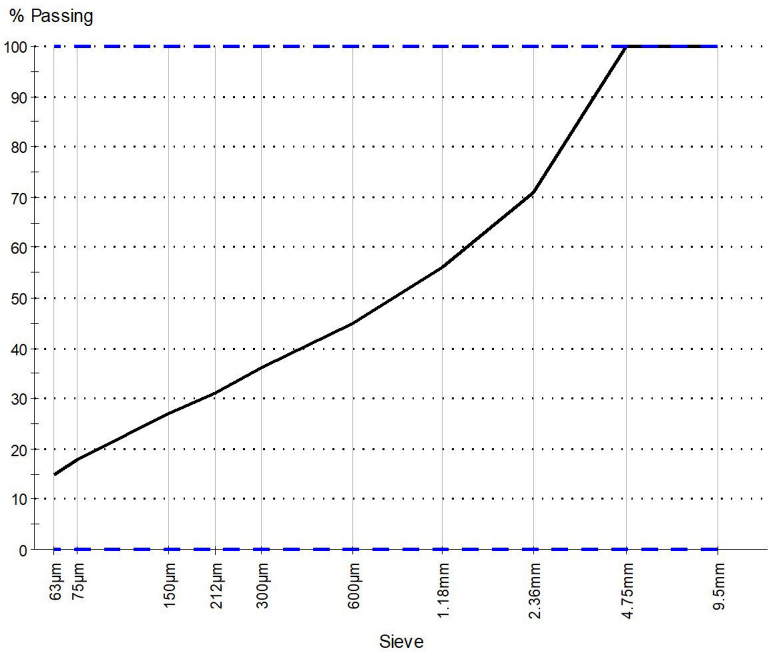
## Sample Details

| | |
|---|---|
| **Sample ID:** | CTS26S-00605 |
| **Client Sample ID:** | 09698 |
| **Material:** | Crushed Pea Gravel |
| **Sample Source:** | Central - Parkburn Quarry |
| **Site/Sampled From:** | Pembroke Heights, Wanaka |
| **Date Sampled:** | 20/02/2026 |
| **Specification:** | No Specification |
| **Sampled By:** | Nevan Danischewski |
| **Sampling Method:** | NZS 4407:2015 2.4.8.3 (EX PAVEMENT/SEL) |
| **Technician:** | Henry Clatworthy |
| **Sampling Endorsed?:** | Yes |

## Other Test Results

| Description | Method | Result | Limits |
|---|---|---|---|

## Particle Size Distribution




**Method:** NZS 4402:1986 Test 2.8.1 - Modified
**Drying By:** Oven
**Date Tested:** 11/03/2026
**Tested By:** Henry Clatworthy

| Sieve Size | % Passing | Limits |
|---|---|---|
| 9.5mm | 100 | 0 – 100 |
| 4.75mm | 100 | 0 – 100 |
| 2.36mm | 71 | 0 – 100 |
| 1.18mm | 56 | 0 – 100 |
| 600µm | 45 | 0 – 100 |
| 300µm | 36 | 0 – 100 |
| 212µm | 31 | 0 – 100 |
| 150µm | 27 | 0 – 100 |
| 75µm | 18 | 0 – 100 |
| 63µm | 15 | 0 – 100 |

## Comments

Date Received: 20-Feb-26

**Central Testing Services**
18 Ngapara St, P.O. Box 397, Alexandra 9340, Central Otago, New Zealand
P: 03 4487644, W: www.centraltesting.co.nz, E: info@centraltesting.co.nz



**Reference No: CTS26W0575-1**

**Date: 7 April 2026**

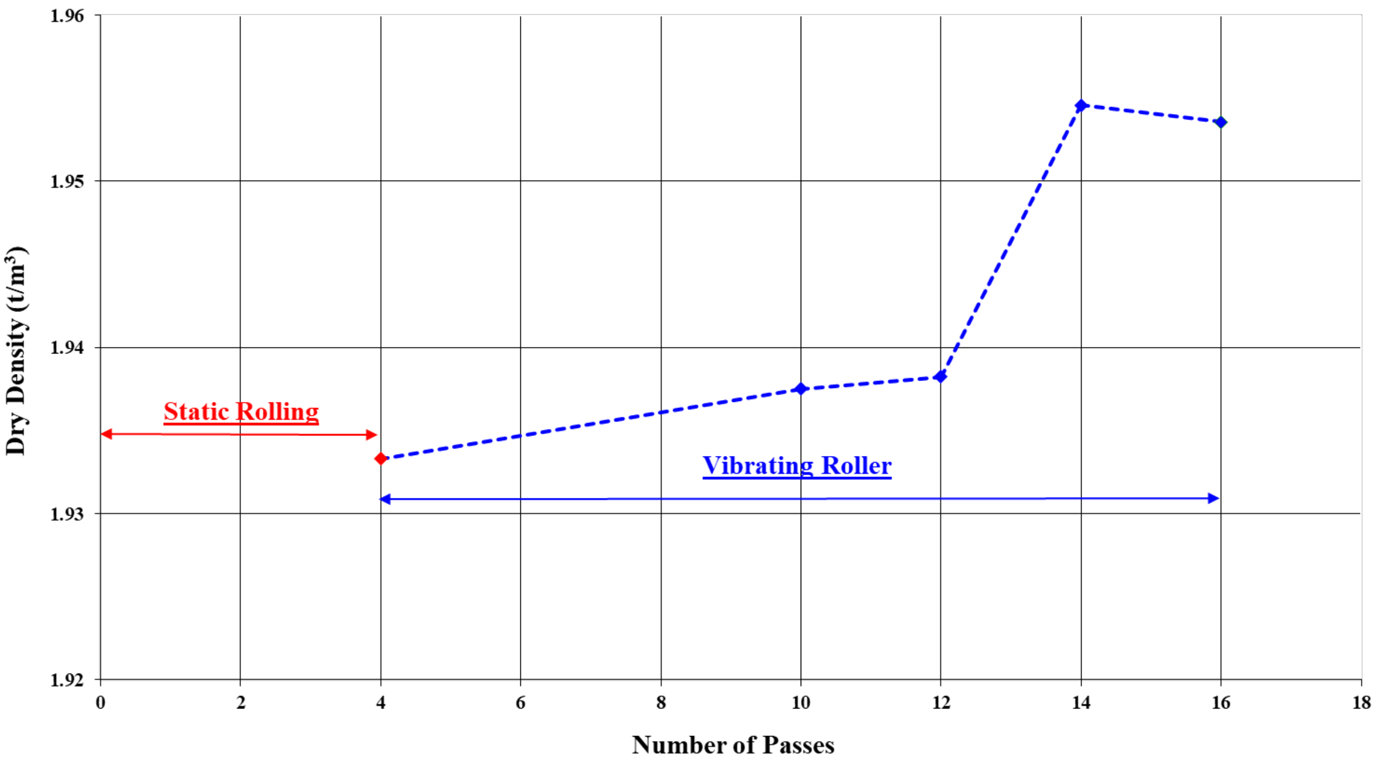
# *TEST REPORT – PLATEAU DENSITY & WATER CONTENT*

| | | | |
|---|---|---|---|
| **Client Details:** | **Insight Engineering, P.O. Box 456, Cromwell** | **Attention:** | **J. Kruyshaar** |
| **Job Description:** | **Pembroke Heights, Wanaka – Compaction Trial #2, Layer 1** | | |
| **Sample Description:** | **Crushed Pea Gravel** | **Sample Source:** | **Parkburn** |
| **Sample Method:** | **NZS 4407:2015, Test 2.4.8.3** | **Sampled By:** | **N.P. Danischewski** |
| **Test Methods:** | **Field Density - NZS 4407:2015, Test 4.2 (Direct transmission - 250mm probe depth);<br>Water Content - NZS 4402:1986, Test 2.1** | | |

| Compaction Used (13T Bomag) | Total Passes | Wet Density ($t/m^3$) | Dry Density ($t/m^3$) | Corrected Water Content (%) | Relative Compaction (%) | Air Voids (%) | Total Voids (%) |
|---|---|---|---|---|---|---|---|
| 4 Passes Static | 4 | 1.99 | 1.93 | 3.1 | 94 | 22 | 28 |
| 4 Passes Static + 6 Passes Vib | 10 | 2.01 | 1.94 | 3.9 | 94 | 21 | 28 |
| 4 Passes Static + 8 Passes Vib | 12 | 2.01 | 1.94 | 3.8 | 94 | 21 | 28 |
| 4 Passes Static + 10 Passes Vib | 14 | 2.04 | 1.95 | 4.1 | 95 | 19 | 28 |
| 4 Passes Static + 12 Passes Vib | 16 | 2.04 | 1.95 | 4.2 | 95 | 19 | 28 |

*Note: The density results reported above are the mean of 3 individual NDM probes (excluding 4 passes static – one reading only). 1 pass = 1 direction.*

## PLATEAU DENSITY




*General Notes:*

- *Relative Compaction values have been calculated using a maximum dry density of 2.06 $t/m^3$. Voids have been calculated using an assumed solid density of 2.70 $t/m^3$ (See Reference No. CTS26W0577 for NZ standard compaction details).*
- *A water content correction based on comparative laboratory water contents has been applied to this data.*
- *This report may not be reproduced except in full.*

**Tested By:** **N.P. Danischewski & A. Rowe** **Date: 20 to 22-Feb-26** **Checked By:** A. Rowe

**Approved Signatory**

A. Rowe

**A. Rowe**
**Key Technical Personnel**

ACCREDITED TESTING LABORATORY

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

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**Reference No:** CTS26W0575-2

**Date:** 7 April 2026

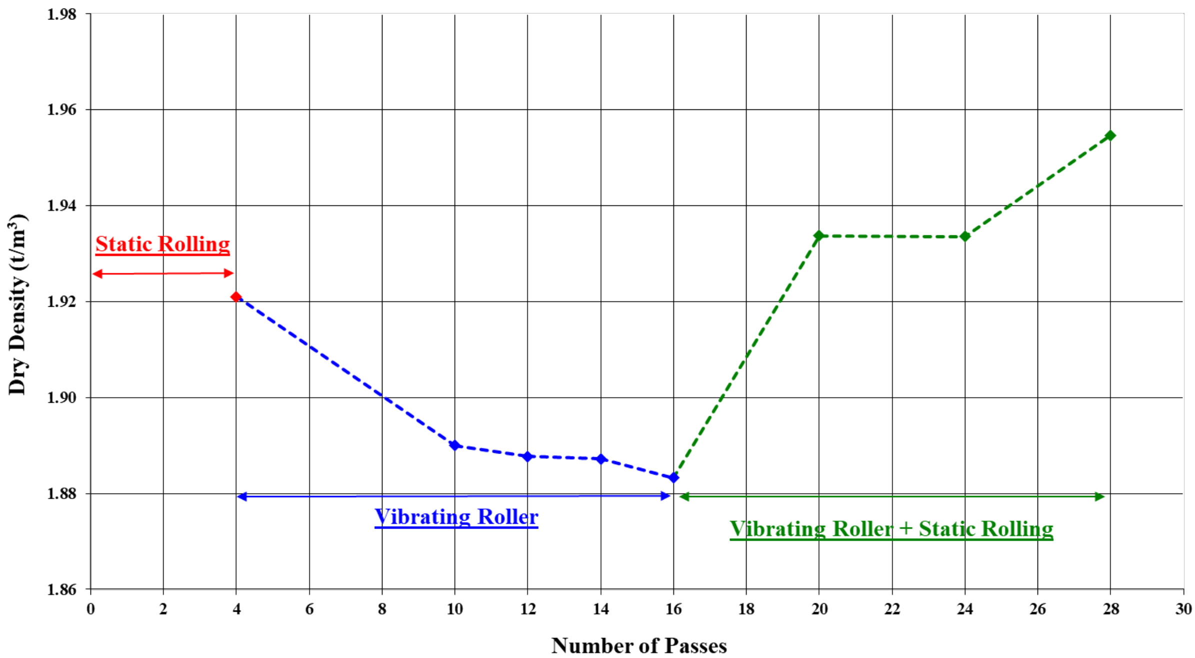
# TEST REPORT – PLATEAU DENSITY & WATER CONTENT

| | | | |
|---|---|---|---|
| **Client Details:** | Insight Engiueering, P.O. Box 456, Cromwell | **Attention:** | J. Kruyshaar |
| **Job Description:** | Pembroke Heights, Wanaka – Compaction Trial #2, Layer 1 | | |
| **Sample Description:** | Crushed Pea Gravel | **Sample Source:** | Parkburn |
| **Sample Method:** | NZS 4407:2015, Test 2.4.8.3 | **Sampled By:** | N.P. Danischewski |
| **Test Methods:** | Field Density - NZS 4407:2015, Test 4.3 (backscatter method); Water Content - NZS 4402:1986, Test 2.1 | | |

| Compaction Used (13T Bomag) | Total Passes | Wet Density ($t/m^3$) | Dry Density ($t/m^3$) | Corrected Water Content (%) | Relative Compaction (%) | Air Voids (%) | Total Voids (%) |
|---|---|---|---|---|---|---|---|
| 4 Passes Static | 4 | 1.99 | 1.92 | 3.6 | 93 | 22 | 29 |
| 4 Passes Static + 6 Passes Vib | 10 | 1.98 | 1.89 | 4.7 | 92 | 21 | 30 |
| 4 Passes Static + 8 Passes Vib | 12 | 1.98 | 1.89 | 4.7 | 92 | 21 | 30 |
| 4 Passes Static + 10 Passes Vib | 14 | 1.97 | 1.89 | 4.6 | 92 | 22 | 30 |
| 4 Passes Static + 12 Passes Vib | 16 | 1.97 | 1.88 | 4.5 | 91 | 22 | 30 |
| 8 Passes Static + 12 Passes Vibe | 20 | 2.03 | 1.93 | 4.9 | 94 | 19 | 28 |
| 12 Passes Static + 12 Passes Vibe | 24 | 2.03 | 1.93 | 5.1 | 94 | 19 | 28 |
| 16 Passes Static + 12 Passes Vibe | 28 | 2.05 | 1.95 | 4.8 | 95 | 18 | 28 |

*Note: The density results reported above are the mean of 3 individual NDM probes (excluding 4 passes static – one reading only).*
*1 pass = 1 direction.*

## PLATEAU DENSITY




*General Notes:*

- *Relative Compaction values have been calculated using a maximum dry density of 2.06 $t/m^3$. Voids have been calculated using an assumed solid density of 2.70 $t/m^3$ (See Reference No. CTS26W0577 for NZ standard compaction details).*
- *A water content correction based on comparative laboratory water contents has been applied to this data.*
- *This report may not be reproduced except in full.*

**Tested By:** N.P. Danischewski & A. Rowe **Date:** 20 to 22-Feb-26 **Checked By:** A. Rowe

**Approved Signatory**

A. Rowe

**A. Rowe**
**Key Technical Personnel**

ACCREDITED IANZ TESTING LABORATORY № 434

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

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P: 03 4487644, W: www.centraltesting.co.nz, E: info@centraltesting.co.nz



**Reference No: CTS26W0575-3**

**Date: 7 April 2026**

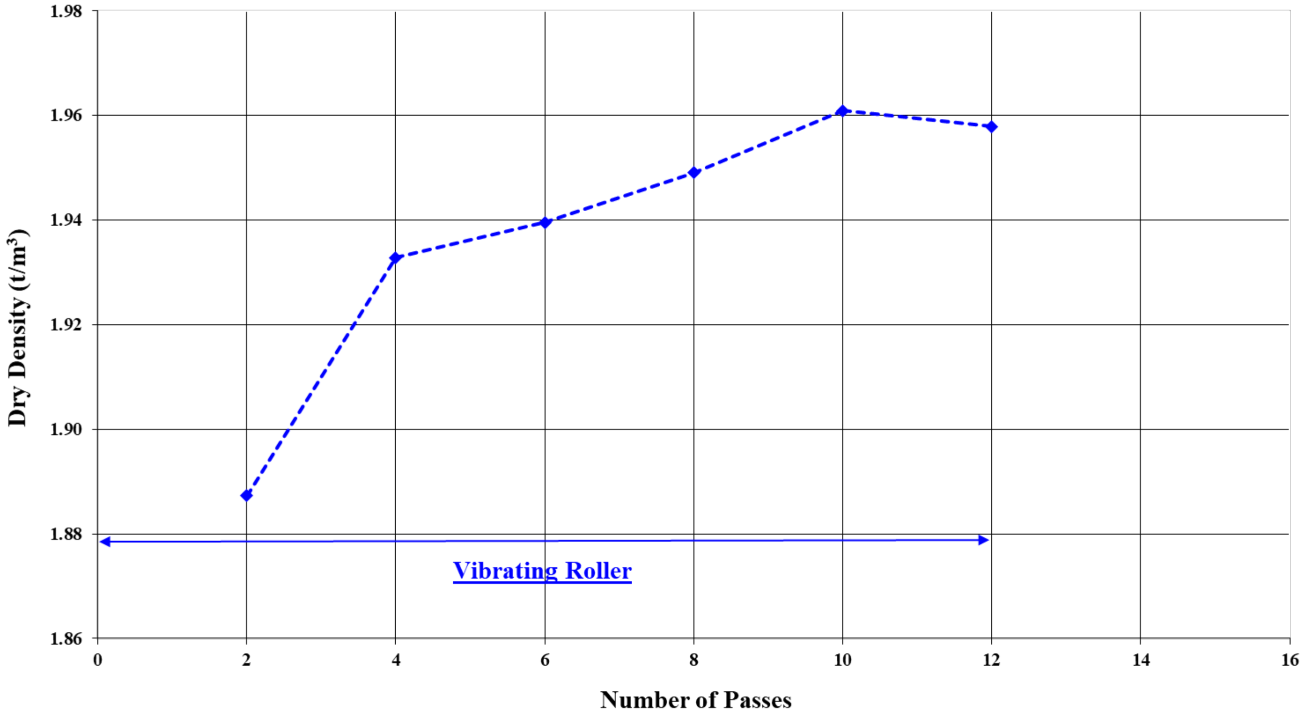
# *TEST REPORT – PLATEAU DENSITY & WATER CONTENT*

| | | | |
|---|---|---|---|
| **Client Details:** | **Insight Engineering, P.O. Box 456, Cromwell** | **Attention:** | **J. Kruyshaar** |
| **Job Description:** | **Pembroke Heights, Wanaka – Compaction Trial #2, Layer 2** | | |
| **Sample Description:** | **Crushed Pea Gravel** | **Sample Source:** | **Parkburn** |
| **Sample Method:** | **NZS 4407:2015, Test 2.4.8.3** | **Sampled By:** | **N.P. Danischewski** |
| **Test Methods:** | **Field Density - NZS 4407:2015, Test 4.2 (Direct transmission - 250mm probe depth);<br>Water Content - NZS 4402:1986, Test 2.1** | | |

| Compaction Used (13T Bomag) | Total Passes | Wet Density ($t/m^3$) | Dry Density ($t/m^3$) | Corrected Water Content (%) | Relative Compaction (%) | Air Voids (%) | Total Voids (%) |
|---|---|---|---|---|---|---|---|
| 2 Passes Vib | 2 | 1.95 | 1.89 | 3.4 | 92 | 24 | 30 |
| 4 Passes Vib | 4 | 2.01 | 1.93 | 3.7 | 94 | 21 | 28 |
| 6 Passes Vib | 6 | 2.01 | 1.94 | 3.9 | 94 | 21 | 28 |
| 8 Passes Vib | 8 | 2.02 | 1.95 | 3.7 | 95 | 21 | 28 |
| 10 Passes Vib | 10 | 2.03 | 1.96 | 3.6 | 95 | 20 | 28 |
| 12 Passes Vib | 12 | 2.03 | 1.96 | 3.7 | 95 | 20 | 27 |

*Note: The density results reported above are the mean of 3 individual NDM probes.*
*1 pass = 1 direction.*

## PLATEAU DENSITY




*General Notes:*

- *Relative Compaction values have been calculated using a maximum dry density of 2.06 $t/m^3$. Voids have been calculated using an assumed solid density of 2.70 $t/m^3$ (See Reference No. CTS26W0577 for NZ standard compaction details).*
- *A water content correction based on comparative laboratory water contents has been applied to this data.*
- *This report may not be reproduced except in full.*

**Tested By:** **N.P. Danischewski & A. Rowe** **Date: 20 to 22-Feb-26** **Checked By:** A. Rowe

**Approved Signatory**

A. Rowe

**A. Rowe**
**Key Technical Personnel**

ACCREDITED TESTING LABORATORY

**All tests reported herein have been performed in accordance with the laboratory's scope of accreditation**

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18 Ngapara St, P.O. Box 397, Alexandra 9340, Central Otago, New Zealand
P: 03 4487644, W: www.centraltesting.co.nz, E: info@centraltesting.co.nz



**Reference No:** CTS26W0575-4

**Date:** 7 April 2026

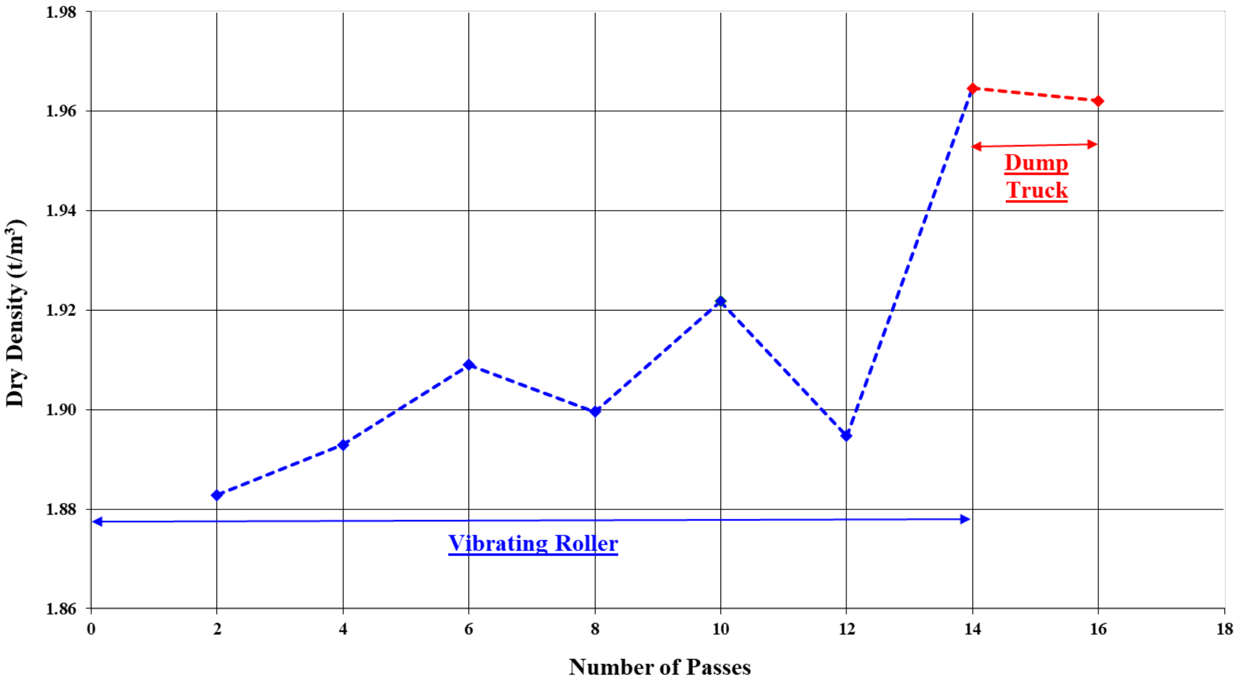
# TEST REPORT – PLATEAU DENSITY & WATER CONTENT

| | | | |
|---|---|---|---|
| **Client Details:** | Insight Engiueering, P.O. Box 456, Cromwell | **Attention:** | J. Kruyshaar |
| **Job Description:** | Pembroke Heights, Wanaka – Compaction Trial #2, Layer 2 | | |
| **Sample Description:** | Crushed Pea Gravel | **Sample Source:** | Parkburn |
| **Sample Method:** | NZS 4407:2015, Test 2.4.8.3 | **Sampled By:** | N.P. Danischewski |
| **Test Methods:** | Field Density - NZS 4407:2015, Test 4.3 (backscatter method); Water Content - NZS 4402:1986, Test 2.1 | | |

| Compaction Used (13T Bomag) | Total Passes | Wet Density ($t/m^3$) | Dry Density ($t/m^3$) | Corrected Water Content (%) | Relative Compaction (%) | Air Voids (%) | Total Voids (%) |
|---|---|---|---|---|---|---|---|
| 2 passes vibe | 2 | 1.93 | 1.88 | 2.6 | 91 | 25 | 30 |
| 4 passes vibe | 4 | 1.95 | 1.89 | 2.9 | 92 | 24 | 30 |
| 6 passes vibe | 6 | 1.97 | 1.91 | 3.2 | 93 | 23 | 29 |
| 8 passes vibe | 8 | 1.95 | 1.90 | 2.8 | 92 | 24 | 30 |
| 10 passes vibe | 10 | 1.98 | 1.92 | 3.0 | 93 | 23 | 29 |
| 12 passes vibe | 12 | 1.95 | 1.89 | 2.9 | 92 | 24 | 30 |
| 12 passes vibe + 2 passes dump truck | 14 | 2.03 | 1.96 | 3.4 | 95 | 21 | 27 |
| 12 passes vibe + 4 passes dump truck | 16 | 2.03 | 1.96 | 3.4 | 95 | 21 | 27 |

*Note: The density results reported above are the mean of 3 individual NDM probes.*
*1 pass = 1 direction.*

## PLATEAU DENSITY




*General Notes:*

- *Relative Compaction values have been calculated using a maximum dry density of 2.06 $t/m^3$. Voids have been calculated using an assumed solid density of 2.70 $t/m^3$ (See Reference No. CTS26W0577 for NZ standard compaction details).*
- *A water content correction based on comparative laboratory water contents has been applied to this data.*
- *This report may not be reproduced except in full.*

**Tested By:** N.P. Danischewski & A. Rowe **Date:** 20 to 22-Feb-26 **Checked By:** A. Rowe

**Approved Signatory**

A. Rowe

**A. Rowe**
**Key Technical Personnel**

ACCREDITED IANZ TESTING LABORATORY №434

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

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**Reference No:** CTS6W0577

**Date: 7 April 2026**

# TEST REPORT - NZ STANDARD COMPACTION

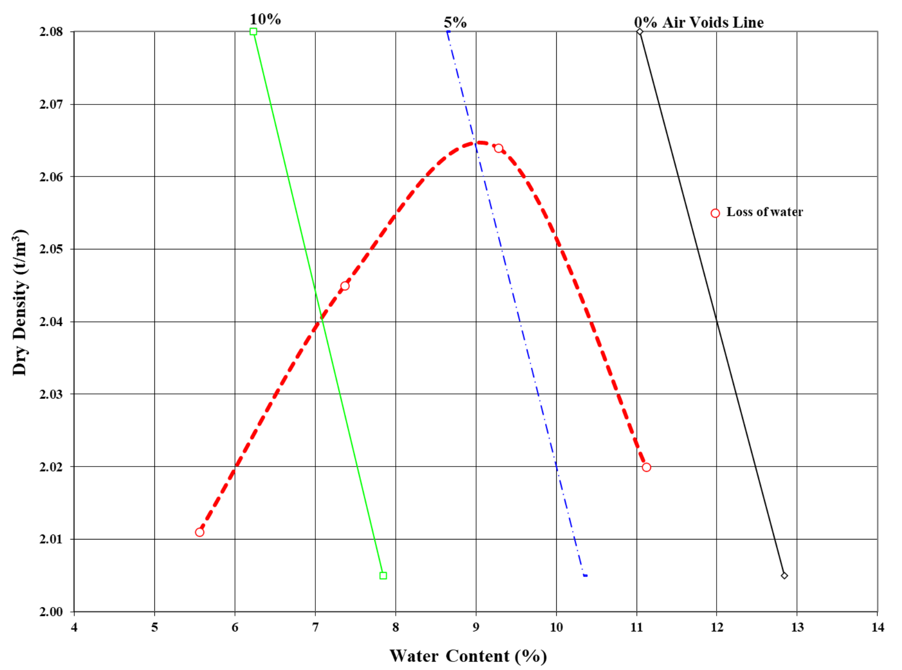
| | | | |
|---|---|---|---|
| **Client Details:** | **Insight Engineering, P.O. Box 456, Cromwell** | **Attention:** | **J. Kruyshaar** |
| **Job Description:** | **Pembroke Heights** | | |
| **Sample Description:** | **Crushed Pea Gravel** | **Order No:** | **N/A** |
| **Sample Source:** | **Parkburn** | **Sample Label No:** | **09698** |
| **Date & Time Sampled:** | **20-Feb-26** | **Sampled By:** | **N.P. Danischewski** |
| **Sample Method:** | **NZS 4407:2015, Test 2.4.8.3** | **Date Received:** | **20-Feb-26** |

## NZ STANDARD COMPACTION - NZS 4402:1986, Test 4.1.1

| | |
|---|---|
| **% Retained (+19.0mm Fraction)** | **0.0 %** |
| **Maximum Dry Density:** | **$2.06 t/m^3$** |
| **Optimum Water Content:** | **9.0 %** |

*Notes:*

- *The sample was received in a natural state.*
- *The material tested in the NZ Standard Compaction test was whole soil.*
- *The air voids lines were calculated from an assumed solid density of $2.70\ t/m^3$.*




*General Notes:*

- *This report may not be reproduced except in full.*

**Tested By:** **M. Duncan** **Date:** **27 to 31-Mar-26**

**Checked By:**

**Approved Signatory**

**N.P. Danischewski**
**Key Technical Personnel**

**All tests reported herein have been performed in accordance with the laboratory's scope of accreditation**

ACCREDITED TESTING LABORATORY № 434

# Appendix 5

## Draft Fill Specifications

# Draft Bulk Earthworks Specification for Park Burn

**Specification for Local Borrow materials**

The bulk earthworks shall be undertaken in accordance with NZS4431:2022 unless noted otherwise in this report and specification.

| | | | | |
|---|---|---|---|---|
| **Material definition** | **Fill use type** | Bulk 4431 fill | | |
| | **Source material type** | C-A (coarse-grained – acceptable, GAP100 pit run) | | |
| | **Typical use description** | General fill for residential/commercial earthworks using site-won materials | | |
| **Source material acceptance testing**[3] | **Test and method** | **Minimum test frequency** | **Normal acceptance criteria** | **Notes** |
| | Particle size distribution (NZS4407 3.8 or NZS4402 2.8.1) | 1 for each source and 1 for each change in material | Refer to Table A2 NZS4431:2022 | Completed before works begin and during the works if material changes |
| | Dry density/water content relationship (NZS 4402 4.1.3 or 4.2.1, 4.2.2 and 4.2.3) | 1 for each source and 1 for each change in material | OMC and MDD determined for compaction acceptance testing | Completed before works begin and during the works if material changes. Not required for coarse-grained soils where continuous compaction control will be used to assure compaction quality. |
| | Water Content (NZS4402 3.1) | 1 for each source and 1 for each change in material | Between OMC -2% and OMC +4% | Ongoing during the works confirm conditioning requirements. Not use if there is no well-defined density/water content relationship |
| | Solid density of soil particles (NZS4402 test 2.7) | 1 for each source and 1 for each change in material | Solid density determined for in-situ density testing | Can be omitted where the geotechnical engineer can provide a reliable estimate for the specific source material or where alternative compaction acceptance testing is proposed. |
| **Placement requirements**[1] | Loose layer thickness < 250 mm<br>Wetting as necessary | | | |
| **Compaction acceptance testing** | **Test and method** | **Minimum test frequency** | **Normal acceptance criteria** | **Notes** |
| | For coarse grained soils with max particle size <65mmm: Field water content and density (NZS4402 2.1<br>NZS4407 4.1,4.2 & 4.3) | 1 per 1,000 $m^3$ (minimum 2 per lift[4]) | ≥95% MDD where fill height is <3 m<br><br>≥98% MDD where fill height is >3m<br><br><15% air voids | **Acceptance criteria is subject to Dry Density/Water content test method. 92% is acceptable for Vibrating Hammer method.** |
| | For coarse grained soil with max particle size 65-250mm, proof roll to target calibrated dynamic response modulus or relative density (CEN/TS 17006) | Ongoing continuous compaction control | Target minimum calibrated compaction machine specific dynamic response modulus or minimum 80% relative density | Target minimum modulus set by correlation with plate load test(s) or as otherwise defined by the certifier |
| | Dynamic Cone Penetrometer | 2 per 1,000 $m^3$ for full depth of lift (minimum 2 per lift) | ≥ 5 blows per 100 mm | The DCP is indicative only and should not be used alone for compaction compliance. |
| | Impact test – 4.5 kg hammer (ASTM D5874) | 1 per 50 m3 on each compacted layer (min 2 for each lift of fill) | IV>25 | This test is indicative only and should not be used alone for compaction compliance. |
| | Plate load test (DIM 18134) | As specified by the certifier | ≥300 kPa ultimate bearing capacity<br><25 mm settlement at 300 kPa | Typical frequency 1 per lot at completion of final lift |

[3] Refer to Laboratory test results (attached)
[4] For this project the maximum height of a lift is taken as no greater than 750 mm or three compacted layers

# Draft Bulk Earthworks Specification for Park Burn

**Specification for Crushed Pea Gravel (Impact crusher)**

The bulk earthworks shall be undertaken in accordance with NZS4431:2022 unless noted otherwise in this report and specification.

| | | | | |
|---|---|---|---|---|
| **Material definition** | **Fill use type** | Bulk 4431 fill | | |
| | **Source material type** | F-D (fine-grained – dry) | | |
| | **Typical use description** | General fill for residential/commercial earthworks using crushed pea gravel | | |
| **Source material acceptance testing**[5] | **Test and method** | **Minimum test frequency** | **Normal acceptance criteria** | **Notes** |
| | Particle size distribution (NZS4402 2.8.1) | 1 for each source and 1 for each change in material | Refer to Table A2 NZS4431:2022 | Completed before works begin and during the works if material changes |
| | Dry density/water content relationship (NZS 4402 2.1 & 4.1.1) | 1 for each source and 1 for each change in material | OMC and MDD determined for compaction acceptance testing | Completed before works begin and during the works if material changes |
| | Water Content (NZS4402 2.1) | 1 for each source and 1 for each change in material | Between OMC -2% and OMC +4% | Ongoing during the works confirm conditioning requirements |
| | Liquid and plastic limit (NZS4402 2.2, 2.3 & 2.4) | 1 for each source and 1 for each change in material | Plasticity index < 25%<br>Liquid limit <50% | Completed before works begin and during the works if material changes. Only if high silt content. |
| **Placement requirements**[1] | Loose layer thickness < 500 mm<br>Wetting as necessary | | | |
| **Compaction acceptance testing** | **Test and method** | **Minimum test frequency** | **Normal acceptance criteria** | **Notes** |
| | Field water content and density (NZS4402 2.1 NZS4407 4.1,4.2 & 4.3) | 2 per 1,000 m3 (minimum 2 per lift[6]) | ≥95% MDD where fill height is <3 m<br>≥98% MDD where fill height is >3m<br><15% air voids | Nuclear Density Meter on direct transmission or backscatter mode. |
| | Dynamic Cone Penetrometer | As specified by the certifier (maximum depth of test is 2 m) | ≥ 5 blows per 100 mm | The DCP is indicative only and should not be used alone for compaction compliance. |
| | Impact test – 4.5 kg hammer (ASTM D5874) | 1 per 50 m3 on each compacted layer (min 2 for each lift of fill) | IV>25 | This test is indicative only and should not be used alone for compaction compliance. |

[5] Refer to Compaction Trial Plateau test results (attached)
[6] For this project the maximum height of a lift is taken as no greater than 1000 mm or three compacted layers

**Central Testing Services**

18 Ngapara St, P.O. Box 397, Alexandra 9340, Central Otago, New Zealand

P: 03 4487644, W: www.centraltesting.co.nz, E: info@centraltesting.co.nz



**Reference No:** **CTS25W2031**

**Date: 16 June 2025**

# TEST REPORT - NZ VIBRATING HAMMER COMPACTION

| | | | |
|---|---|---|---|
| **Client Details:** | Fulton Hogan Land Development Ltd, Level 1 Annex, 152 Oxford Street, Christchurch | **Attention:** | T. James |
| **Job Description:** | Parkburn Investigations | | |
| **Sample Description:** | Pit Run - Cobbly GRAVEL with minor sand & trace of silt | **Client Order No:** | N/A |
| **Sample Source:** (cs) | Central – Parkburn Quarry | **Sample Label No:** | 05510 |
| **Date & Time Sampled:** (cs) | 3-Jun-25 | **Sampled By:** (cs) | G. Marsh & J. Ford |
| **Sample Method:** (cs) | NZS 4407:2015, Test 2.4.6.3.2 (machine stockpile) | **Date Received:** | 3-Jun-25 |

**MAXIMUM DRY DENSITY, OPTIMUM WATER CONTENT & DRY DENSITY RESULTS**

| | |
|---|---|
| **% Retained: (+37.5mm Fraction)** | 28.0 % |
| **Dry Density: (+37.5mm Fraction)** | 2.68 $t/m^3$ |
| **Absorption: (+37.5mm Fraction)** | 0.5 % |
| **Maximum Dry Density: (-37.5mm Fraction)** | 2.34 $t/m^3$ |
| **Optimum Water Content: (-37.5mm Fraction)** | 2.6 % |

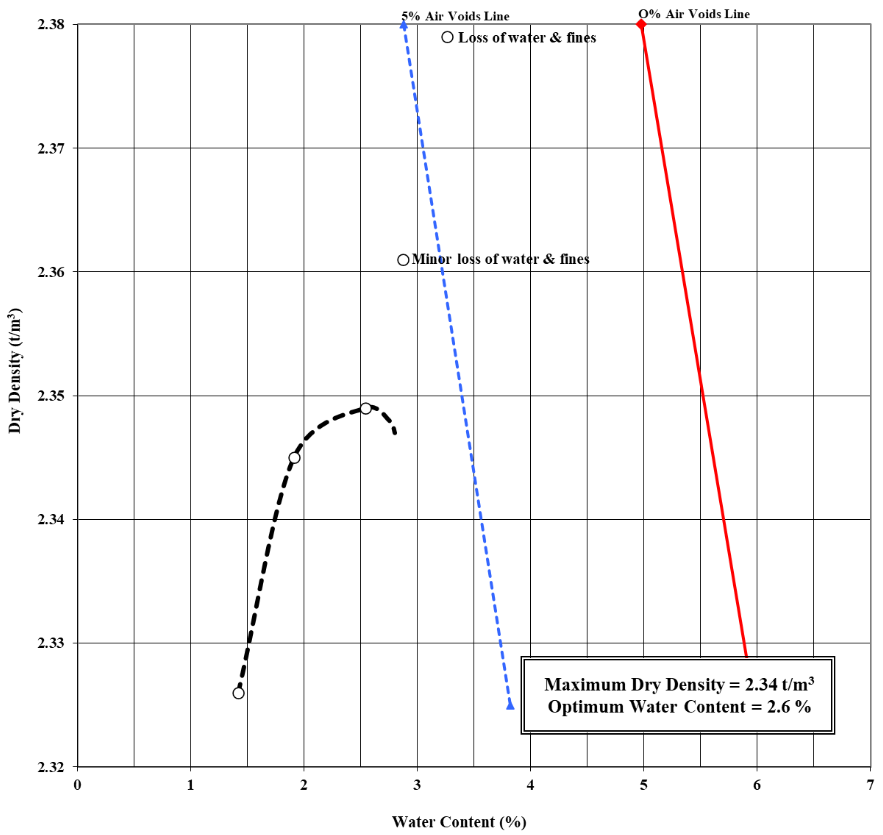
Test Methods:

Dry Density: NZS 3111:1986, Test 12

Maximum Dry Density: NZS 4402:1986, Test 4.1.3

*Notes:*

- *The material was received in a natural state.*
- *NZS 4402:1986, Test 4.1.3 states the maximum dry density is to be reported to the nearest 0.02 $t/m^3$. The maximum dry density reported to the nearest 0.01 $t/m^3$ = 2.35 $t/m^3$ (Not IANZ Accredited).*
- *The sample tested was the fraction passing a 37.5mm test sieve.*
- *The air voids lines were calculated from an assumed solid density of 2.70 $t/m^3$.*




*Additional Notes:*

- *Information contained in this report which is Not IANZ Accredited relates to the sample description based on NZ Geotechnical Society Guidelines 2005, the client supplied information (cs) and sampling.*
- *This report may not be reproduced except in full.*

**Tested By:** J. Smith **Date:** 9 & 10-Jun-25 **Checked By:**

Approved Signatory

C. Julius
Key Technical Personnel `

ACCREDITED TESTING LABORATORY № 434

Test results indicated as not accredited are outside the scope of the laboratory's accreditation

**Work Order ID:** CTS25W1915

**Central Testing Services**
18 Ngapara Street, Alexandra 9340, Central Otago
P.O Box 397, Alexandra, Central Otago
**Telephone:** +64 3 4487644
**www.centraltesting.co.nz**

# Material Test Report

**Report No: MAT:CTS25S-01765**
**Issue No: 1**

**Client:** Cant Land Resource
Corner Main South & Halswell Junction Rd
Hornby

Christchurch 8042
NZ

**Project:** Parkburn Investigations

The tests reported herein (unless otherwise indicated) have been performed in accordance with the laboratory's scope of accreditation. Samples are tested as received, in natural condition, unless stated otherwise in the comments. This report may only be reproduced in full.

The results in this report relate only to the items / samples that were tested

Approved Signatory: Carl Julius
(Laboratory Technician)
IANZ Accreditation No:434
Date of Issue: 6/06/2025

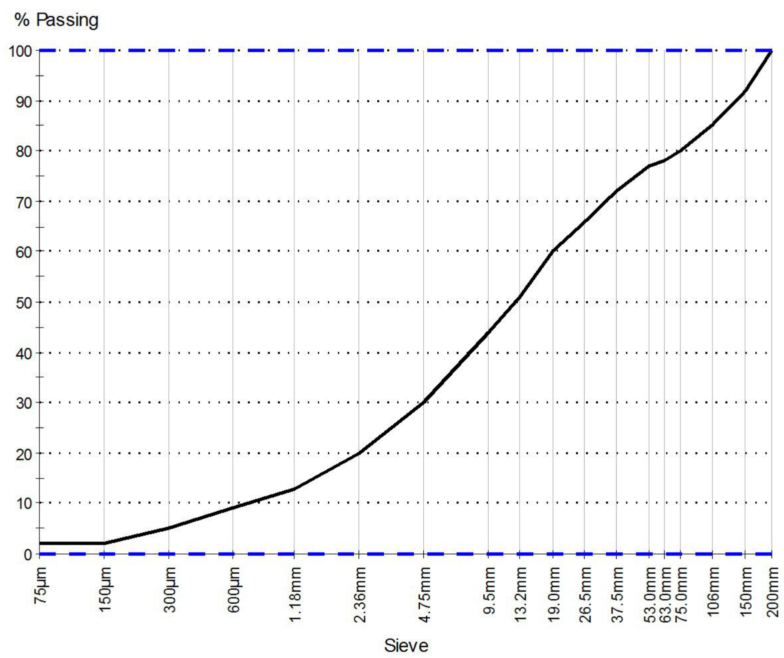
## Sample Details

| | |
|---|---|
| **Sample ID:** | CTS25S-01765 |
| **Client Sample ID:** | 09674 |
| **Material:** | Pit Run - Cobbly GRAVEL with minor sand and trace of silt |
| **Sample Source:** | Central - Parkburn Quarry |
| **Site/Sampled From:** | Stockpile |
| **Date Sampled:** | 26/05/2025 |
| **Specification:** | No Specification |
| **Sampled By:** | Cam Maxwell |
| **Sampling Method:** | NZS 4407:2015 2.4.6.3.2 (SP/WG/MACHINE) |
| **Technician:** | Adam Rowe |
| **Sampling Endorsed?:** | Yes |

## Other Test Results

| Description | Method | Result | Limits |
|---|---|---|---|

## Particle Size Distribution




**Method:** NZS 4407:2015 Test 3.8.1
**Drying By:** Oven
**Date Tested:** 30/05/2025
**Tested By:** Adam Rowe
**Note:** [a] Obtained by difference.

| Sieve Size | % Passing | Limits |
|---|---|---|
| 200mm | 100 | 0 – 100 |
| 150mm | 92 | 0 – 100 |
| 106mm | 85 | 0 – 100 |
| 75.0mm | 80 | 0 – 100 |
| 63.0mm | 78 | 0 – 100 |
| 53.0mm | 77 | 0 – 100 |
| 37.5mm | 72 | 0 – 100 |
| 26.5mm | 66 | 0 – 100 |
| 19.0mm | 60 | 0 – 100 |
| 13.2mm | 51 | 0 – 100 |
| 9.5mm | 44 | 0 – 100 |
| 4.75mm | 30 | 0 – 100 |
| 2.36mm | 20 | 0 – 100 |
| 1.18mm | 13 | 0 – 100 |
| 600µm | 9 | 0 – 100 |
| 300µm | 5 | 0 – 100 |
| 150µm | 2 | 0 – 100 |
| 75µm | 2[a] | 0 – 100 |

## Comments

Date Received: 27-May-25

**Central Testing Services**
18 Ngapara Street, Alexandra 9340, Central Otago
P.O Box 397, Alexandra, Central Otago
**Telephone:** +64 3 4487644
**www.centraltesting.co.nz**

Central Testing Services

# Material Test Report

**Report No: MAT:CTS25S-01793**
**Issue No: 1**

**Client:**
Cant Land Resource
Corner Main South & Halswell Junction Rd
Hornby

Christchurch 8042
NZ

**Project:** Parkburn Investigations

ACCREDITED IANZ TESTING LABORATORY

The results in this report relate only to the items / samples that were tested

The tests reported herein (unless otherwise indicated) have been performed in accordance with the laboratory's scope of accreditation. Samples are tested as received, in natural condition, unless stated otherwise in the comments. This report may only be reproduced in full.

Approved Signatory: Carl Julius
(Laboratory Technician)
IANZ Accreditation No:434
Date of Issue: 18/06/2025

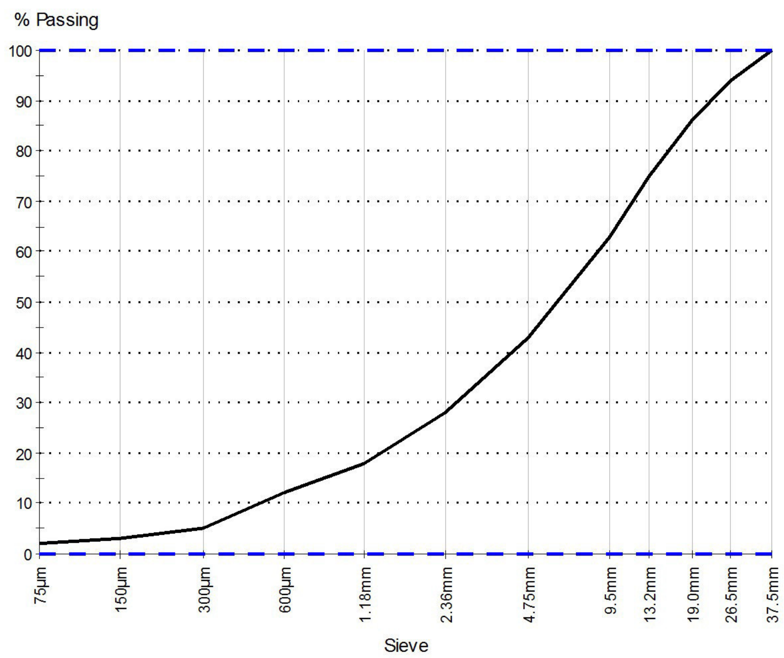
## Sample Details

**Sample ID:** CTS25S-01793
**Client Sample ID:** 05510
**Material:** Pit Run - GRAVEL with some sand and trace of silt
**Sample Source:** Central - Parkburn Quarry
**Site/Sampled From:** Parkburn
**Date Sampled:** 03/06/2025
**Specification:** No Specification
**Sampled By:** Gary Marsh
**Sampling Method:** NZS 4407:2015 2.4.6.3.2 (SP/WG/MACHINE)
**Technician:** Adam Rowe
**Sampling Endorsed?:** No

## Other Test Results

| Description | Method | Result | Limits |
|---|---|---|---|

## Particle Size Distribution




**Method:** NZS 4407:2015 Test 3.8.1
**Drying By:** Oven
**Date Tested:** 13/06/2025
**Tested By:** Adam Rowe
**Note:** [a] Obtained by difference.

| Sieve Size | % Passing | Limits |
|---|---|---|
| 37.5mm | 100 | 0 – 100 |
| 26.5mm | 94 | 0 – 100 |
| 19.0mm | 86 | 0 – 100 |
| 13.2mm | 75 | 0 – 100 |
| 9.5mm | 63 | 0 – 100 |
| 4.75mm | 43 | 0 – 100 |
| 2.36mm | 28 | 0 – 100 |
| 1.18mm | 18 | 0 – 100 |
| 600µm | 12 | 0 – 100 |
| 300µm | 5 | 0 – 100 |
| 150µm | 3 | 0 – 100 |
| 75µm | 2[a] | 0 – 100 |

## Comments

Note: Material is estimated to have 35% all in Pea Gravel.
Date Received: 3-Jun-25

**Work Order ID:** CTS25W2033

**Central Testing Services**
18 Ngapara Street, Alexandra 9340, Central Otago
P.O Box 397, Alexandra, Central Otago
**Telephone:** +64 3 4487644
**www.centraltesting.co.nz**

**Report No: MAT:CTS25S-01797**
**Issue No: 1**

# Material Test Report

**Client:**
Cant Land Resource
Corner Main South & Halswell Junction Rd
Hornby

Christchurch 8042
NZ

**Project:** Parkburn Investigations

The tests reported herein (unless otherwise indicated) have been performed in accordance with the laboratory's scope of accreditation. Samples are tested as received, in natural condition, unless stated otherwise in the comments. This report may only be reproduced in full.

The results in this report relate only to the items / samples that were tested

Approved Signatory: Carl Julius
(Laboratory Technician)
IANZ Accreditation No:434
Date of Issue: 18/06/2025

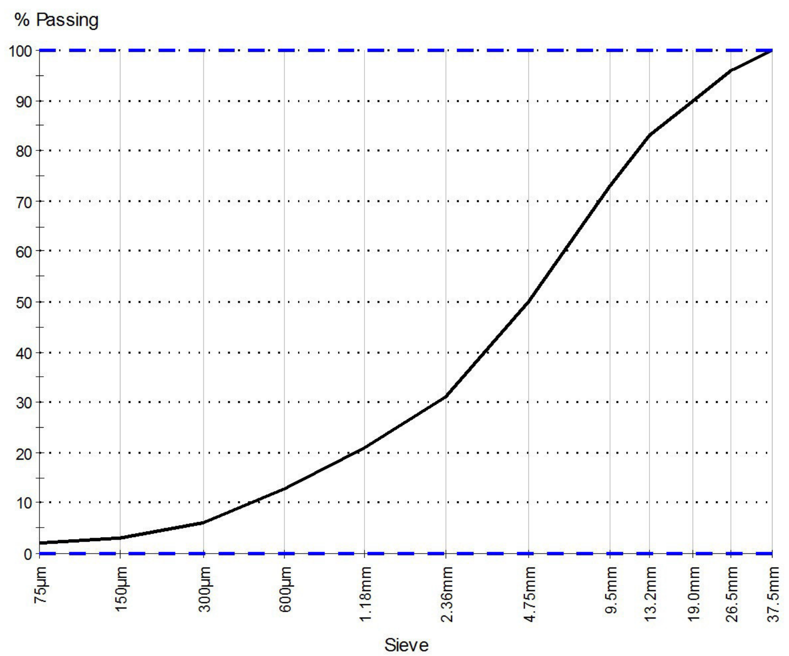
## Sample Details

| | |
|---|---|
| **Sample ID:** | CTS25S-01797 |
| **Client Sample ID:** | 05510 |
| **Material:** | Pit Run - GRAVEL with some sand and trace of silt |
| **Sample Source:** | Central - Parkburn Quarry |
| **Site/Sampled From:** | Parkburn |
| **Date Sampled:** | 03/06/2025 |
| **Specification:** | No Specification |
| **Sampled By:** | Gary Marsh |
| **Sampling Method:** | NZS 4407:2015 2.4.6.3.2 (SP/WG/MACHINE) |
| **Technician:** | Adam Rowe |
| **Sampling Endorsed?:** | No |

## Other Test Results

| Description | Method | Result | Limits |
|---|---|---|---|

## Particle Size Distribution




**Method:** NZS 4407:2015 Test 3.8.1
**Drying By:** Oven
**Date Tested:** 13/06/2025
**Tested By:** Adam Rowe
**Note:** [a] Obtained by difference.

| Sieve Size | % Passing | Limits |
|---|---|---|
| 37.5mm | 100 | 0 – 100 |
| 26.5mm | 96 | 0 – 100 |
| 19.0mm | 90 | 0 – 100 |
| 13.2mm | 83 | 0 – 100 |
| 9.5mm | 73 | 0 – 100 |
| 4.75mm | 50 | 0 – 100 |
| 2.36mm | 31 | 0 – 100 |
| 1.18mm | 21 | 0 – 100 |
| 600µm | 13 | 0 – 100 |
| 300µm | 6 | 0 – 100 |
| 150µm | 3 | 0 – 100 |
| 75µm | 2[a] | 0 – 100 |

## Comments

Note: Material is estimated to have 50% all in Pea Gravel
Date Received: 3-Jun-25

**Central Testing Services**

18 Ngapara St, P.O. Box 397, Alexandra 9340, Central Otago, New Zealand

P: 03 4487644, W: www.centraltesting.co.nz, E: info@centraltesting.co.nz



**Reference No:** **CTS6W0577**

**Date: 7 April 2026**

# *TEST REPORT - NZ STANDARD COMPACTION*

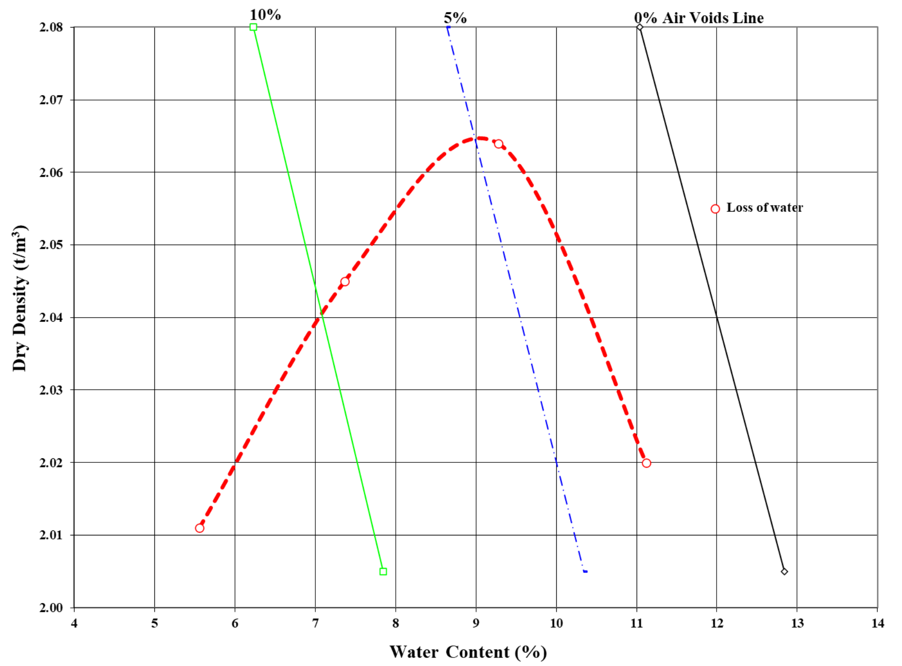
| | | | |
|---|---|---|---|
| **Client Details:** | **Insight Engineering, P.O. Box 456, Cromwell** | **Attention:** | **J. Kruyshaar** |
| **Job Description:** | **Pembroke Heights** | | |
| **Sample Description:** | **Crushed Pea Gravel** | **Order No:** | **N/A** |
| **Sample Source:** | **Parkburn** | **Sample Label No:** | **09698** |
| **Date & Time Sampled:** | **20-Feb-26** | **Sampled By:** | **N.P. Danischewski** |
| **Sample Method:** | **NZS 4407:2015, Test 2.4.8.3** | **Date Received:** | **20-Feb-26** |

## NZ STANDARD COMPACTION - NZS 4402:1986, Test 4.1.1

| | |
|---|---|
| **% Retained (+19.0mm Fraction)** | **0.0 %** |
| **Maximum Dry Density:** | **2.06t/m$^3$** |
| **Optimum Water Content:** | **9.0 %** |

*Notes:*

- *The sample was received in a natural state.*
- *The material tested in the NZ Standard Compaction test was whole soil.*
- *The air voids lines were calculated from an assumed solid density of 2.70 t/m$^3$.*




*General Notes:*

- *This report may not be reproduced except in full.*

**Tested By:** **M. Duncan** **Date:** **27 to 31-Mar-26**

**Checked By:**

**Approved Signatory**

**N.P. Danischewski**
**Key Technical Personnel**

**All tests reported herein have been performed in accordance with the laboratory's scope of accreditation**

ACCREDITED TESTING LABORATORY №434