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RESIN INJECTION GROUND IMPROVEMENT RESEARCH TRIALS

AVONDALE AND BEXLEY RED ZONE CHRISTCHURCH

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Resin Injection Ground Improvement Trials

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1 Introduction

Resin injection has long been used internationally for the lifting or 'level correction' of buildings. For this purpose, expanding resin mixes are injected into the ground at relatively shallow depths, resulting in ground heave and building lift. It is logical to expect that this resin injection and expansion process also results in compaction or densification of the ground (in a similar way to compaction grouting). As such, resin injection can potentially be used to mitigate liquefaction or for other applications where ground densification is required.

Figure 1 Schematic, resin injection using the Mainmark Teretek™ system as a level correction method

The aim of this study is to examine whether resin injection is a viable form of ground improvement, primarily for liquefaction mitigation. This has been achieved through a series of trial injection panels located at three different sites, where pre-improvement and post-improvement soil density and stiffness have been compared via cone penetration testing (CPT), direct push crosshole testing (V_s and V_p testing) and dilatometer testing (DMT). The results of other relatively recent testing and commercial application of resin injection for liquefaction mitigation are also discussed in this report (the 2013 EQC ground improvement trials, and a 2015/2016 commercial building project).

This study was largely funded by Mainmark Ground Engineering Ltd (Mainmark), with contributions towards field testing and peer review by MBIE and EQC. Mainmark also provided all equipment, materials, and site management for the project.

2 Report Summary

- At each of three sites in the Christchurch 'Red Zone', trial panels approximately 8m by 8m were constructed by injecting resin to 6m depth on a 1.2m by 1.2m triangular grid. (Section 6.2)
- Soil testing by CPT, cross hole geophysics, and DMT was carried out on both the untreated and treated soils at each site. (Section 6.2)
- To model typical building loads, each panel was surcharged during resin injection using concrete block kentledge to apply a 14 kPa general surcharge as well as a 17 kN/m foundation line load. (Section 8.1)
- There was notable and consistent increase in the density of the sandy soils (including the siltier sands – e.g., sands with CPT I_C values up to about 2.0) at all three sites. Post-improvement CPT tip resistance has increased on average 80%, resulting in an average increase in q_{c1Ncs} in the order of 70%. (Section 9.1)
- Calculated post-improvement liquefaction settlements and LSN (liquefaction severity number) values have reduced by 50 – 80%. (Section 9.3, 9.4)
- Measured post-improvement cross-hole shear wave velocities (Vs) have increased in the order of 40%, resulting in the soil shear stiffness increasing by approximately a factor of two. (Section 9.1)
- DMT indicates increases in parameter KD of about 80%, which implies potential further increases in the resistance of the soils to liquefaction due to increased horizontal stresses. (Section 9.5)
- The consistent increases in post-improvement soil density and stiffness (to a point at which liquefaction triggering is either eliminated or greatly reduced within the target improvement zone) indicates that the implied surface damage potential due to liquefaction is significantly reduced. (Section 9.4)
- Computed post-improvement static bearing capacities for shallow foundations increased by approximately 20 to 75%, and static settlements decreased by approximately 15 -60%. (Section 9.6)
- The effectiveness of improvement increases with both decreasing fines content (particularly below an I_C of 2.0), and increasing confining pressures, as would be expected for a densification method. The former will constrain the applicability of this technology to any given site, however the latter can be controlled with the application of kentledge. (Section 10)
- The resin injection methodology does not appear to loosen soils at low confining stresses, unlike compaction grouting methods (EQC, in prep). Nonetheless, the potential for relatively minor ground heave needs to be taken into account, and is largely controllable. (Section 3, 8.4)

- Resin injection can be applied to cleared sites, however it is particularly useful in situations where a structure already exists and requires improvement of the ground beneath it (e.g. for liquefaction mitigation, or bearing capacity enhancement), where there are limited alternative options. (Section 4.2)

- The results of this study demonstrate that resin injection is a viable ground improvement method for mitigation of liquefaction potential, and also for increasing foundation bearing capacities in sandy soils. (Section 11)

3 Soil Improvement Method and Mechanism

The primary mechanism of improvement is densification of the soil due to aggressive expansion of the polyurethane resin product. Secondary effects such as improvement in composite stiffness, cementation, and horizontal stress increases are also present.

With this method, injection tubes are driven into the ground on a regular spacing (typically on a triangular to square pattern), and an injection nozzle is attached to the injection tube. Multipart materials are mixed at specific pressures and temperatures at the nozzle, and the live composite material ('resin') is then pumped down to the base of the tube, where it enters the soil matrix. Either 'top down' or 'bottom up' methods can be employed. In a typical 'bottom up' installation the tube is installed to a target depth and then withdrawn either in set stages with set volumes of material injected at each stage, or it is slowly withdrawn at a uniform rate, with set volumes of material being injected per unit length of withdrawal. This is more efficient in terms of time and injection hardware (i.e. tubing) than top-down injection. However, top-down injection has the advantage of providing a 'capping' layer to reduce the tendency of near-surface injected resin materials to migrate towards the ground surface. A hybrid of the two methods was used in this trial. (The choice of method can be site specific, and depends on factors such as soil types and density, access, degree of improvement required, location within the soil column of the soils to be improved, and time constraints).

The low viscosity resin is injected at controlled pressures and penetrates the soil mass along pre-existing planes of weakness or through fracturing of the soil mass. The resin also permeates the soil mass to a limited extent; depending on the porosity of the soil. The resin mix chemically reacts soon after injection (at controllable 'rise' times), rapidly expanding to many times it original volume and changing from a fluid form to a solid one. The expansion volume can be in the order of 5 – 15 times injected volume, or more if required, depending on soil density, confinement pressure, and the resin material selected. The looser the soil, the greater the expansion for a given resin mix.

The expansion of the injected material results in compaction of the adjacent soils, due to new material being introduced into a relatively constant soil volume. The injection/expansion process can result in some soil heave at the ground surface although this can be controlled through injection depth as well as control of the reaction/expansion characteristics (both volume and timing) of the resin materials, or the use of kentledge where appropriate. This process is also often used at shallower depths beneath foundations and slabs to lift and relevel buildings, particularly in finer grained soils. This aspect is further discussed in Section 8.4.

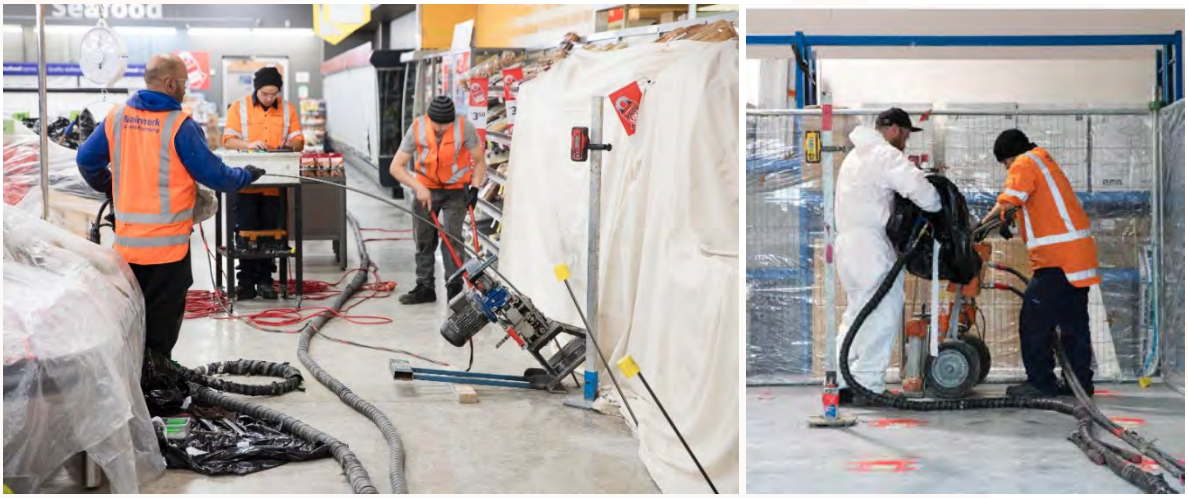


Figure 2 Installing grout tubes (left) and injecting resin (right) inside a supermarket (see Section 4.2)

It should be noted that the expanding resin injection process does not produce regular columns or 'bulbs' of material distributed down the vertical injection line. Instead it typically results in a 'veining' of expanded material distributed throughout the soil mass as dykes, sills or networks of sheets or plates, typically tens of millimetres thick (refer to Figures 3 & 4).

Figure 3 Shallow hand-exhumed resin veins, site 3 spacing trial panel.

Figure 4 Hydro-exhumed resin veins, site 3 spacing trial panel – showing more extensive resin structure.

Visible in Figures 3 and 4 is a series of veins or dykes that are sub-parallel to the adjacent Avon River (some 90 metres distant), and also a second orthogonal set of resin veins. These two sets of veins typically intersect at the resin injection points to form a semi-cellular structure. It is possible that the set of veins that is running sub-parallel to the river is as a result of the resin

following planes of weakness or zones of reduced horizontal stress, resulting from lateral spreading movement of the ground towards the river during the damaging earthquakes of the 2010 – 2011 Canterbury Earthquake Sequence. The orthogonal set of veins is more difficult to explain – however post-earthquake crack mapping by the New Zealand Earthquake Commission (EQC) shows a slight tendency for development of two sets of orthogonal crack patterns in this neighbourhood (the more obvious set being sub-parallel to the river).

As discussed in the introduction in Section 1, the densification mechanism is similar to compaction grouting, however there are some differences. Low mobility grout (LMG), which is a compaction grouting method tested in the 2013 EQC trials, uses a 'thick' cement-based grout that is pumped into the ground under relatively high pressure. The grout was generally observed to follow pre-existing planes of weakness (e.g., fractures, bedding planes) and tended to lengthen fissures already present. At low confining stresses, this process was found during the EQC trials to result in the loosening of the ground in several instances.

Injected resin, in contrast to cement grout, remains as a (low viscosity) fluid for a very short period of time (normally a matter of seconds) before entering a rapid expansion phase which tends to occur volumetrically in all directions, rather than only increasing the length of any planes of weakness or fissures that might be present. The combination of low viscosity injection followed by extremely rapid and large expansion appears to largely control the loosening of the ground at low confining stresses.

4 Background and Recent Use

Ground strengthening by resin injection has been previously used in Turkey, as reported on by Erdemgil et al (2007). Liquefaction mitigation by resin injection in New Zealand has been examined in some detail on two recent occasions. Testing of resin injection was carried out as part of the EQC Ground Improvement trials in 2013, and it was also used on a relatively large commercial building rehabilitation project in late 2015 / early 2016. The method is currently being considered for several other commercial strengthening projects.

4.1 EQC Ground Improvement Trials, 2013

A series of ground improvement trials were undertaken in Christchurch in the latter half of 2013, to examine the performance of various forms of shallow ground improvement for liquefaction mitigation. The trials included a limited examination of resin injection, which is discussed in more detail in Attachment 1. In summary, the resin injection panel that was tested showed an increase in liquefaction resistance via a number of measures (refer to Figure 5). Generally the overall density of the soil increased, as measured by CPT tip resistance.

Truck-mounted vibroseis (aka "T-Rex") testing of the resin injection test panel showed that this improvement method reduced cyclic shear strains relative to the adjacent natural soil panel by nearly an order of magnitude. The decrease in development of cyclic shear strains in the resin test panel corresponded to significant (~25 to 30%) increases in V_s within the injection zone. Ground surface settlement, including differential settlement, due to blast-induced liquefaction was also much reduced.

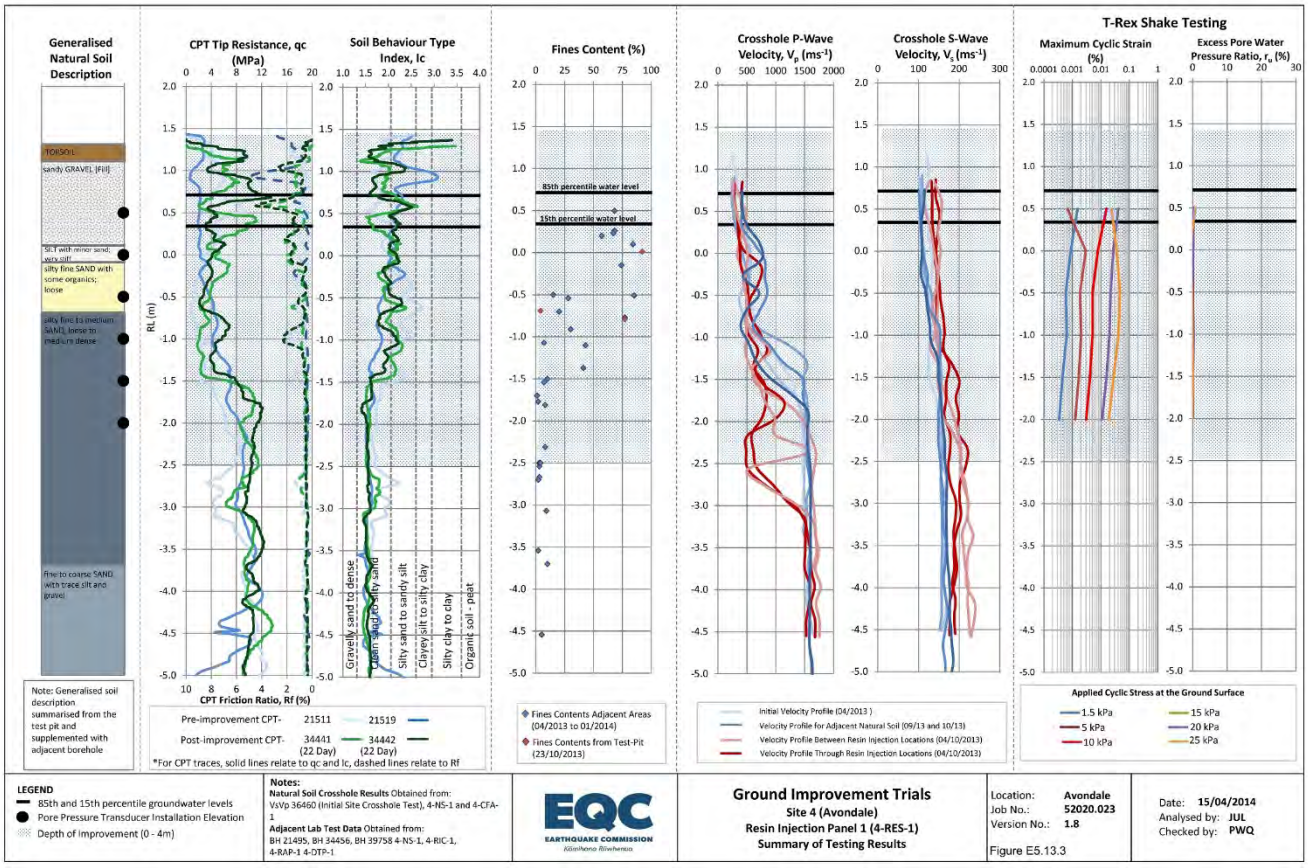


Figure 5 Summary of test results on resin injection panel from EQC draft report. (See Appendix H for a larger version of this figure).

The EQC ground improvement trials report (*in prep*) notes that resin injection was included late in the test programme to assess whether it could be used to relevel houses without worsening the liquefaction potential of the shallow soils due to loosening as a result of ground heave. For this reason, only one test panel was included in the trials. However, given the notable ground improvement for liquefaction mitigation observed during the ground improvement trials, the report recommended that this technique should be subjected to further study.

4.2 Commercial Shopping Centre

Three adjoining large format retail buildings that suffered liquefaction-related settlement damage in the 2010 -2011 Canterbury Earthquake Sequence (up to 160mm differential settlement across the 90m by 60m combined building footprint) were relevelled, repaired, and upgraded in late 2015 and early 2106. The first stage of the remediation works consisted of liquefaction mitigation by densification and stiffening of the underlying shallower soils (treating variously to 4m or 7m depth) using the resin injection methodology described above.

No additional surcharge loading was applied for this project (i.e., other than that imposed by the weight of the buildings). As this was also a relevelling project, some ground heave was purposely developed in specific areas (with the bulk of the lifting being carried out afterwards using 'JOG', a cement based injection system). In areas not targeted for relevelling, a 10mm heave cut off criteria was specified in foundation areas and 5mm in floor areas.

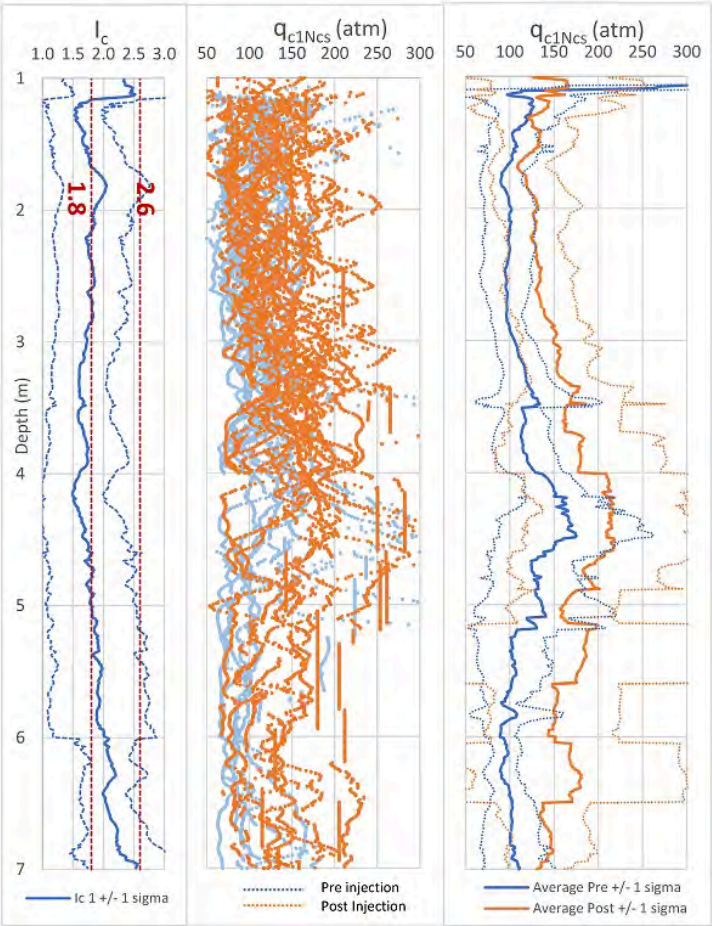


Figure 6: Pre- and post-improvement data from a large commercial shopping centre project

Initial results from test panels constructed at this site are discussed in Traylen, van Ballegooy & Wentz (2016). The subsequent production injection programme resulted in increases in q_{c1ncs} in the order of 40%, (see Figure 6 below) and corresponding decreases in calculated free-field

liquefaction settlements in the treated zones of 40 to 80% under 100 year return period levels of shaking.

The conclusion drawn from this project was that the methodology is a viable technology for ground improvement, and is particularly useful for liquefaction mitigation beneath existing structures. A particular benefit was the low level of intrusion required to carry out the process as the three retail outlets (including a busy supermarket) were able to continue trading, virtually uninterrupted, through the busy Christmas trading period.

This project is discussed in more detail in Attachment 2.

5 Red Zone Site Selection

The series of ground improvement trials undertaken by EQC in Christchurch in the latter half of 2013 were carried out in the Residential 'Red Zone' of Avondale and Bexley in Christchurch. This land was selected because it sustained some of the worst liquefaction-induced land damage during the Canterbury Earthquake Sequence. If a ground improvement method is successful in this 'low quality' land, then it is likely to be successful in most other potentially liquefiable areas (with comparable soil types). It was logical therefore to carry out these resin injection ground improvement trials at the 2013 EQC sites, which were still abandoned and therefore available. This allowed incorporation of some of the characterisation work that had already been carried out for the EQC trials, and also potentially a 'like for like' basis of comparison with the results of those trials (EQC, *in prep*).

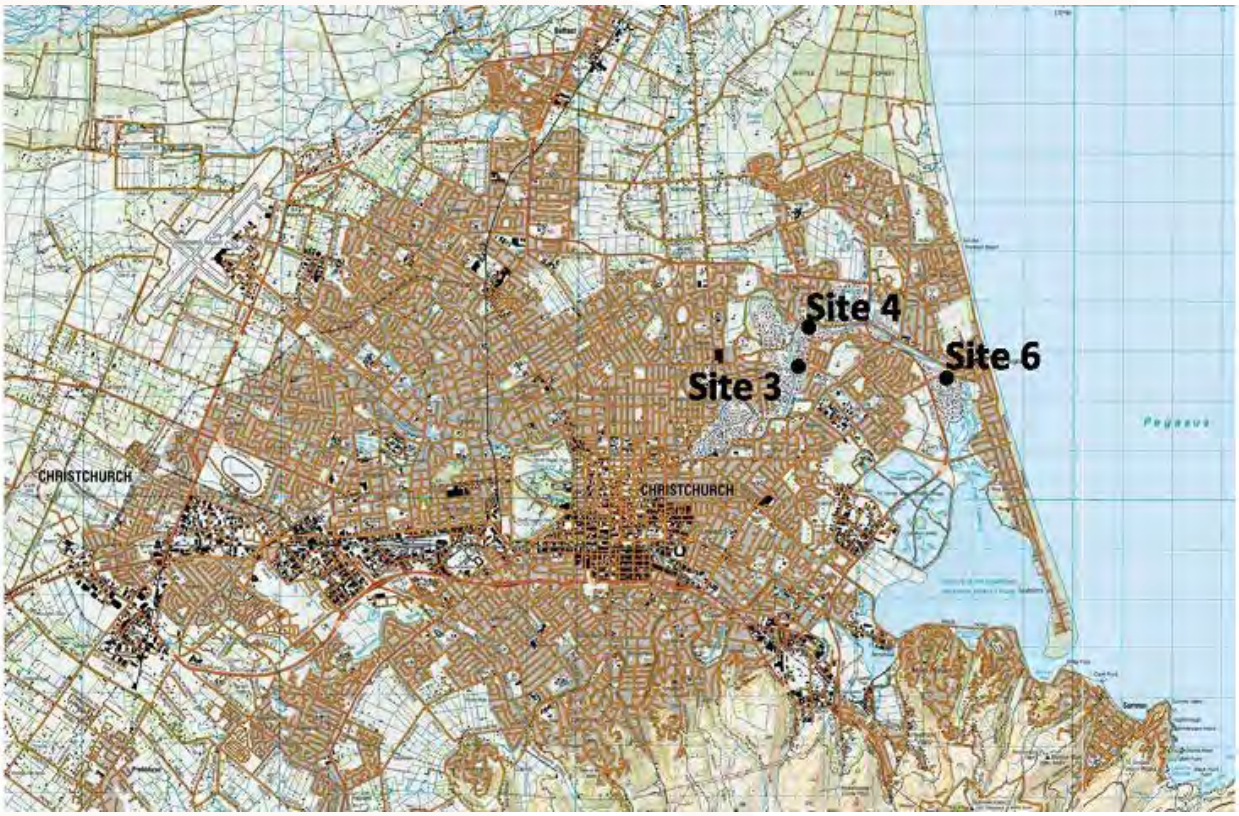


Figure 7 Resin injection test site locations

The resin injection test sites used were located adjacent to three areas used in the EQC trials (refer to Figure 7). The test panels were carefully placed to avoid those areas which had been affected by the installation of previous ground improvements and instruments. They were also located outside of the areas subject to the blast-induced liquefaction trials that were carried out at Site 4 as part of the 2013 EQC work.

The testing at Site 4 in this trial was carried out adjacent to a site that had been blast-tested in 2013, and therefore the ground at the present resin trial site may have been somewhat affected by the blast testing in 2013. However, the current study is based on examining CPT (and other) data measured immediately before the resin injection, and then soon after. The change in the soil state effected by the resin injection (as evidenced by the data) is what is of primary importance, and this is likely to be largely unaffected by the nearby location of the 2013 blasting trials.

See Appendix A for more detailed information on site locations.

6 Test Panel Layouts and Testing Regime

The installation and testing was carried out in two phases, with smaller pre-production 'spacing trial' panels being installed and tested in advance of the main 'production' test panels.

6.1 Trial Test Panels

In advance of the full 'production' test panels, at sites 3 and 6 smaller scale 'spacing trial' test panels were installed by the contractor so that they could refine their installation technique. The primary objective of these trial panels was to confirm the effectiveness of the selected installation technique in the two slightly different ground conditions at the test sites (i.e. sites 3 and 4 are slightly siltier than site 6 – refer to Section 7), so that the production testing at all three sites could be carried out without the need to modify the installation methodology between sites.

The spacing trial panels used concrete blocks to provide a uniform surface loading across the improvement area (to model the presence of a building – refer to Section 8.2). The test results from these panels were successful; yielding promising improvements in measured CPT cone resistance (data included at the back of Appendices E and G), and, in combination with previous test panel results (Traylen et al., 2015) confirming that the effectiveness of the ground improvement is increased by the inclusion of a surface loading. Based on the results of these spacing trials, the contractor selected their preferred production methodology and panel layout.

Figure 8 Spacing trial test panel Site 3 – Injection process underway

6.2 Production Test Panels

Each of the final production panels consisted of a layout approximately 8m by 8m in plan area. The resin injection points were laid out on a 1.2m (centre-centre) triangular grid. Figure 9 shows a typical production test panel layout. The construction of the production panels is discussed in detail in Section 8.

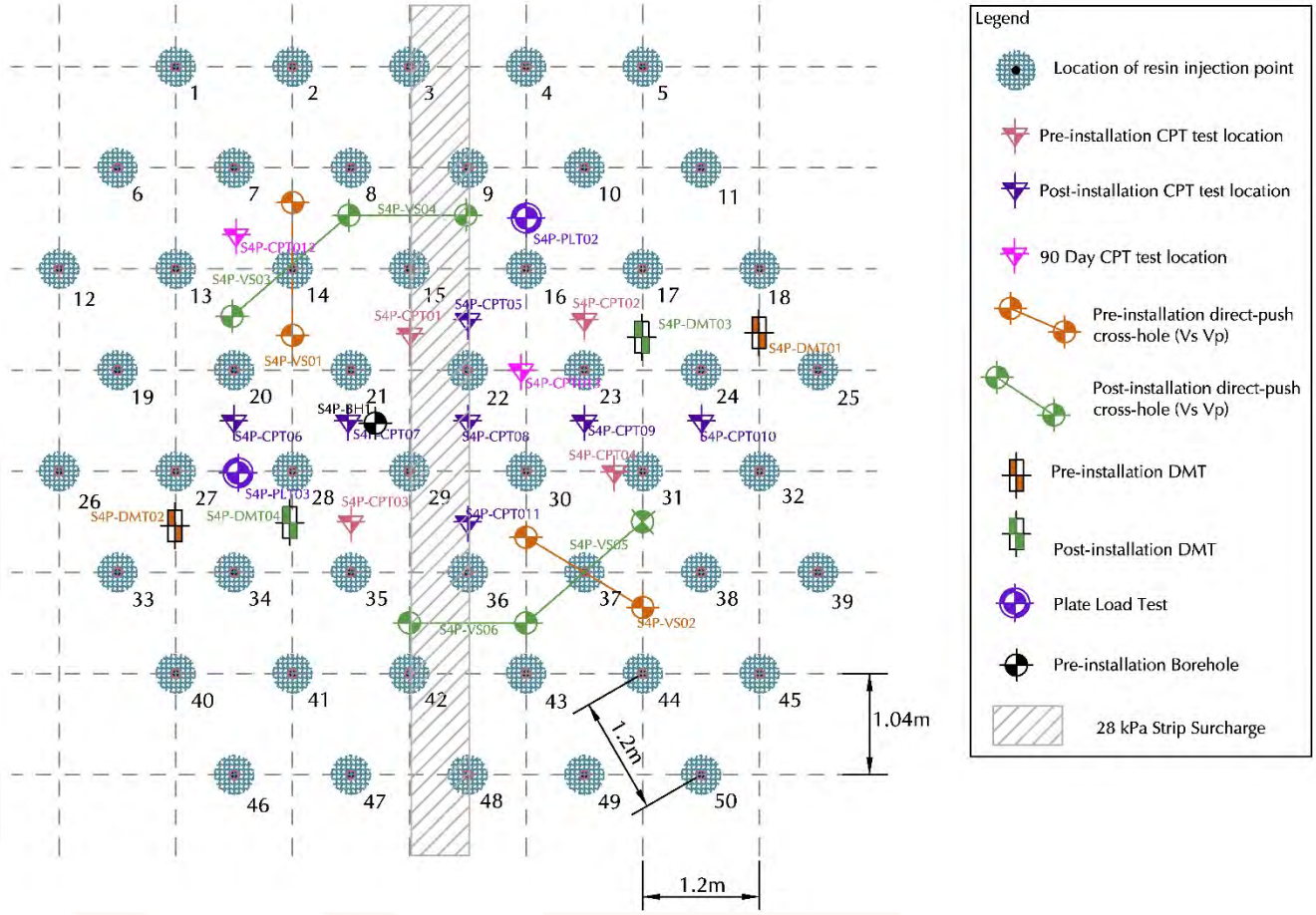


Figure 9 Installation and testing layout Site 4

Pre-improvement in-situ tests and laboratory tests were carried out to both characterise the subsurface soils within each production test panel, and to establish baseline conditions to compare post-improvement testing against. Post-improvement in situ testing was carried out to assess the effectiveness of the expanding resin to densify and stiffen the ground. The in-situ tests were CPT, V_s/V_p, DMT, plate load test (PLT), and borehole (to collect samples for laboratory testing). The number of tests conducted within each test panel are shown in Table 1.

Table 1 Pre and post injection testing

Test	Number carried out at each panel	
	Pre-improvement	Post-improvement
CPT	4	7
V_s/V_P	2	4
DMT tests	2	2
Plate Load Test (PLT)	2	2
Borehole	1	-
Fines content lab test	3-4	-
Plasticity index lab test	0-2	-

The CPT and DMT were used to characterise the soil and measure soil density between injection points. V_s/V_p testing (crosshole testing using the methodology by Wotherspoon et. al., 2016) was performed between the resin injection points to assess the stiffness of the pre-/post-improvement soils, and also across injection points to measure the post-improvement "composite stiffness" of the soil/resin matrix.

CPT, DMT testing, and borehole drilling was carried out by McMillan Drilling Ltd. The V_s/V_p testing was carried out by Dr Liam Wothersoon from the University of Auckland, using LandTest

Ltd rigs to advance the geophones. PLT testing was carried out by Geotechnics Ltd. Central Testing Services Ltd provided the laboratory testing services for the project. The results of the testing for the three production test panels are presented in Appendices E to G.

7 Site Characterisation

This section describes the general subsurface profiles at each of the sites, as well as the groundwater regime.

7.1 Subsurface conditions

From the results of the pre-improvement testing, the following are generalised descriptions of the subsurface conditions at each of the three test sites that were used in this study. It is noted that the soil conditions at the three resin trial sites were largely consistent with those found during the 2013 EQC ground improvement trials, with the exception that resin trial sites 3 and 4 lacked the obvious shallow silt "capping layer" identified during the EQC trials (though the shallow soils still had a notable silt content relative to the deeper soils).

Site 3 – Breezes Road, Avondale

Synopsis: silty sands and some silts overlying clean sands at 3.5-4.5m depth.

Soil conditions consist of grey silty fine sand with occasional silt bands to 3.5-4.5m below ground level. Below this is a fine to coarse grey sand to 8m depth (maximum depth of CPT soundings). Laboratory testing of a silty sand sample in the upper layer showed a fines content of about 40%, while the deeper, cleaner sand layer had fines contents of 1 to 3%. CPT tip resistance (q_c) ranged from about 2 to 5 MPa in the upper 3m, and then approximately 10 to 11 MPa down to 8m depth with the exception of a layer of less dense/soft silty soil between about 5 and 6m that had measured tip resistances of between about 5 and 10 MPa. Cross-hole shear wave velocities averaged 125 – 150 m/s in the upper 3m, and 160 – 175 m/s below that.

Groundwater was measured at a depth of 1.1 to 1.2m (see Section 7.2). The median groundwater depth in the site area, based on modelling by GNS (van Ballegooy, Cox et al., 2014) is 1.1m.

Site 4 – Ardrossan Street, Avondale

Synopsis: silty sands and sandy silts overlying clean sands at 2.5m depth.

Soil conditions consist of interbedded sandy silt and fine silty sand with occasional thin silt layers to about 3 to 4m below ground level. Below this is a fine to coarse sand to about 7m where interbedded siltier layers were present to the maximum depth of investigation of 8m. Laboratory testing of a sandy silt sample at 2m depth showed a fines content of about 80% while a silty sand sample at a depth of 4.2m had a fines content of 45%. The lower sand unit at 5m and 6.5m had a fines content of 2%. CPT q_c values ranged from approximately 1 to 4 MPa in the upper 3 to 4m, and then approximately 10 to 12 MPa down to 7m depth. Between about 7 and 8m, the tip resistance ranged from approximately 4 to 10 MPa. Cross-hole shear wave velocities averaged 140 – 170 m/s in the upper metre of the soil profile, reducing to 115 – 125 m/s to 3.5m depth, and then 130 – 160 m/s below that.

Groundwater was measured at 1.1 to 1.25m depth. The median groundwater depth in the site area, based on modelling by GNS is 1.1m.

Site 6 – Onepu Street, Bexley

Synopsis: a predominantly sandy site.

Soil conditions consist of interbedded sand and silty sand to a depth of about 1.4m below ground level and then a fine to coarse sand to 8m. Laboratory testing of a silty sand

sample at 1.5m depth showed a fines content of about 20%; below 4m the sands had fines contents of 0 to 7%. CPT q_c values increased steadily from 2 to 4 MPa at 1m depth to 10 to 12 MPa at 4m depth. The measured tip resistances decreased somewhat between depths of about 4 and 5m, then steadily increased to about 10 to 14 MPa to the maximum depth investigated of 8m. Cross-hole shear wave velocities averaged 125 – 140 m/s in the upper 3m, and 140 – 175 m/s below that.

Groundwater was measured at 0.65 – 0.85m depth. The median groundwater depth in the site area, based on modelling by GNS is 1.1m.

Appendix C contains borelogs and laboratory test data for all three sites. CPT data can be found (with the other test results) in Appendices E to G.

7.2 Groundwater

A piezometer was installed at each site and the water levels monitored at each site for about 2 weeks, approximately either daily or twice daily. Additionally, the GNS groundwater model (van Ballegooy, Cox et al., 2014) was examined (as published on the New Zealand Geotechnical Database) to determine the median long term groundwater levels at each site. From this a 'design' groundwater depth of 1.0 metre was selected for all three sites. (Given the relatively small spread of results between the sites, it was judged to be preferable, and acceptable, to compare them all against a common groundwater depth for the purposes of this study).

Table 2 Groundwater Depth Summary

Site	Measured Range	GNS Median	Used in Study
3	1.10 – 1.20m	1.10m	1.0m
4	1.10 – 1.25m	1.10m	1.0m
6	0.65 – 0.85m	0.80m	1.0m

[P-wave velocities (V_P) from direct push crosshole testing did indicate partial saturation to depths of 2-5m below ground level at all three sites prior to resin injection. The difference between the depth of measured static groundwater level and depth to 100% saturation has been observed at sites across Christchurch (EQC, in prep.). To take a consistently conservative approach in the assessment of ground improvement however, the depth to fully saturated soil (as determined from Vp testing) was not used in the analyses to limit the layers of soils that were assessed as being prone to liquefaction. Instead, a groundwater depth of 1 metre (assumed to also be the depth to full saturation) was adopted to examine how the observed soil strength data might manifest itself on a typical site in terms of ground deformation or damage post-improvement.]

Appendix D contains groundwater data.

8 Test Panel Construction and Testing

The following sections describe the construction of the test panels, the selection of appropriate ground surcharging loads, and the injection and testing process. It also discusses the ground heave measured during the injection process, and bender-element testing of the resin material itself.

8.1 General

Construction consisted of stripping each site of surficial topsoil, and replacing with 200mm of compacted gravels to provide a stable working surface. After pre-injection CPT, DMT and Vs/Vp testing was then carried out, plywood sheets were laid over the compacted gravels, followed by 600mm high concrete blocks to give a uniform 14 kPa surcharge load across the test panel (refer to Section 8.2 for a discussion on the selection of surcharge loading). Welded steel plate was placed over the concrete blocks to give a stable working platform. Additional blocks were then laid across the centre of each panel to replicate a 17 kN/m strip footing load over a 600mm width, in addition to the 14kPa general surcharge. Pilot holes were drilled and cored through the steel plate, concrete blocks, and plywood to allow the installation of the grout tubes into the ground. The general setup is shown in Figure 10 below.

Figure 10 Site 3 aerial view of production panel resin injection

8.2 Concrete Block Kentledge

The surcharge loads were selected by examining the vertical soil stresses from foundations that might be typical of the buildings that would benefit from the application of this technology. A 2-level unreinforced masonry building ('URM'), with 250mm wide footings applying 50 – 100 kPa loadings to the soils was modelled, as well as a 'supermarket' type commercial building, with 400 – 1000mm wide footings imposing 70 – 175 kPa loading. In each case an assumed 10 kPa

floor load was used. Where this mitigation method is applied to lighter buildings, a kentledge can be applied inside the building if required to enhance the soil response during the injection process.

Elastic solutions yield 10 – 25 kPa vertical stresses beneath and adjacent to footing locations, reducing to 10 kPa some 5m distant from the footing, at 2m depth. At 4m depth these figures are reduced by about 30% within 1m of the modelled footing locations. Modular concrete blocks were locally available that gave about a 14 kPa surcharge to the ground (noting that the 2013 EQC trials used an 8kPa surcharge load during installation). Modelling these concrete blocks along with an additional 28 kPa or 17 kN/m 'footing' strip load yielded results generally in the mid-range of those for the modelled buildings. Therefore the chosen kentledge system is considered a practical and appropriate approximation of foundation/floor loads exerted by typical pre-existing commercial buildings, and also what might be practically achieved in the field for smaller buildings using temporary ballasting systems.

8.3 Ground Treatment and Testing Process

The ground was then treated with injected resin by first applying a top-down 'capping layer' in the upper 1m - 1.5m to provide confinement to the material being injected into the improvement zone. Following injection of the capping layer, resin injection was carried out from 6m depth to 1m depth on a 'bottom up' basis. Set volumes of resin were injected at 0.5m intervals, with the volume being based on the outcomes of the spacing trials and previous experience from the commercial project outlined in Section 4.2. There was no attempt to control ground surface heave in this instance, so no cutoff criteria was set in that regard. (As the concrete blocks were not rigidly attached to each other, they would not behave in a similar manner to an overlying reinforced concrete structure in any case.)

Figure 11 Direct push cross hole geophysical testing at site 4

After a period of at least 2 weeks the concrete blocks were removed, and then post-improvement CPT, V_S/V_P and DMT testing was carried out (refer Section 9 for results). PLT testing was carried out 3 months after the main testing was complete, and additional CPT testing was also carried out at this time (see Sections 9.6 and 9.8 for more details). Therefore all in-situ testing (both pre- and post-improvement) was carried out without the presence of the surcharge loads. The upper about 1.2m of one of the spacing trial panels was also partially exhumed to expose the resin veins in-situ (see Figures 3 and 4 in Section 3).

8.4 Ground Heave

During injection ground heave was observed, as expected. Using a dumpy level, measurements were taken on the top surface of the concrete blocks (or where inaccessible, on the steel plate that capped the first layer of concrete blocks). The measured heave for each of the three production test panels is summarised in Table 3.

Table 3 Measured ground heave

Site	Ground heave		
	Average	Range	Std Dev
3	37mm	10 – 73mm	16mm
4	28mm	7-49mm	10mm
6	70mm	19-135mm	30mm

It was noted that the majority of the lift (e.g. ~70%) occurred during the injection of the capping layer in the upper 1 – 1.5m of the ground profile. The capping layer was installed to restrict the amount of resin material that migrated to the ground surface, rather than expand more usefully in place at greater depth. (This capping layer was not applied in the commercial project discussed in Section 4.2, yet significant densification of the foundation soils was still achieved.)

The pre-improvement soils at site 6 between about 1 and 3m below the ground surface were notably denser than those at sites 3 and 4; hence the greater measured ground heave at site 6 as the capping layer was injected into those soils.

Ground heave would not typically be a concern where this technology is to be applied to cleared sites – although the ground heave and general site disturbance using resin injection is noticeably less than that for other technologies such as stone columns or driven piles. However, ground heave will need to be considered when carrying out ground improvement under existing buildings.

For this project, there was no attempt made to control ground heave or to restrict angular distortions in the overlying blocks, as there was no adverse consequences from heave. Additionally, the overlying blocks do not behave in response to ground movement in the same manner as a continuously reinforced concrete structure, and therefore level differences are somewhat magnified. As a result, relatively large angular distortions did occur. This would not have occurred if heave (which is monitored closely during normal installation) had been controlled by installing additional resin where heave was observed to be the least, and had the overlying kentledge been monolithic.

When injecting resin beneath a building, the amount of ground heave can be relatively well controlled by altering the sequence of injection and/or resin material characteristics, or limiting the amount of material injected to remain within cut-off heave criteria (although this may lead to reductions in the densification achieved). Alternatively, angular distortion can be controlled by temporarily ceasing injection at a point until other adjacent points can be brought up to

similar levels. Building levels can be re-adjusted after resin injection by either further injection, or a complementary cement-based injection relevelling process known as 'JOG'.

In some cases the subject buildings may also require level correction, and therefore some managed ground heave is desirable. In other cases allowances will need to be made so that the subject building can accommodate some changes in final floor and foundation levels. For heavy buildings, or in situations where injection begins at a depth of 2m or more, significant ground heave may not occur at all.

8.5 Resin Bender Element Testing

To ensure that the direct push cross hole geophysical testing (i.e. shear wave velocity testing) was appropriate, the shear wave velocity of a sample of expanded resin was obtained utilising bender element testing in the laboratory at the University of Canterbury. The objective of this testing was to ensure that the shear wave velocity of the injected resin was not too dissimilar to that of the surrounding soil.

As can be seen from the results in Table 4, the shear wave velocity of the resin material is in the order of 300 m/s. Given that the V_s values of the improved soils ranged from about 175 to 275 m/s, the differences between the site profile V_s values and those measured for the resin sample were considered unlikely to adversely affect the cross hole shear wave velocity testing.

Table 4 Bender element test results for resin sample

Confining Pressure	Frequencies tested	Sample height	Shear Wave Velocity
0 kPa	4 kHz, 6 kHz, 8 kHz	98.0 mm	277 m/s
30kPa	4 kHz, 6 kHz, 8 kHz	97.7 mm	310 m/s
50 kPa	4 kHz, 6 kHz, 8 kHz	97.5 mm	305 m/s
100 kPa	4 kHz, 5kHz, 6 kHz, 8 kHz	97.0 mm	297 m/s
50 kPa (unloading)	4 kHz, 6 kHz, 8 kHz	97.0 mm	306 m/s

9 Production Panel Test Results

A clear trend of increased soil density and stiffness is apparent within all three test panels when comparing the results of pre- and post-improvement in-situ testing, with the level of increase generally varying with soil characteristics (i.e. the pre-improvement soil density and fines content). Higher degrees of improvement are particularly noticeable at site 6, where the soils generally have a lower fines content in the upper 3 metres, and the pre-improvement soil densities were higher.

9.1 Overall Results

Detailed test results are shown in Appendices E to G. Figure 12 plots the individual q_{c1Ncs} data for all 3 sites, while summary comparisons of mean pre- and post-improvement CPT tip resistance and soil behaviour type index (I_c), shear wave velocity and DMT vertical drained constrained modulus (M_{DMT}) are presented in Table 5 and in Figures 13 through 15.

Table 5 Increases in averaged parameters within the treatment zone

Site	Percentage Increase in Parameter						
	q_c	q_{c1Ncs}	D_R	V_s			M_{DMT}
				Soil	Composite	Site Average	
3	88%	68%	32%	25%	44%	35%	134%
4	81%	64%	34%	46%	41%	43%	135%
6	101%	76%	27%	50%	52%	51%	221%

The CPT q_c values within the treated zones increased on average 80 to 100% - in the order of 5 MPa at sites 3 and 4, and 10 MPa at site 6. This gave an average increase in normalised equivalent clean sand tip resistance (q_{c1Ncs}) of 65 to 75% (about 50 atm increases at sites 3 and 4, and 100 atm at site 6). Relative density (D_R) increased 30% on average (assessed from the CPT data using the relationship of Jamiolkowski et al. (2001)).

It is noted that there is some variability in the individual results, as Figure 12 below (the pre- and post-improvement qc1ncs data from all three sites) demonstrates:

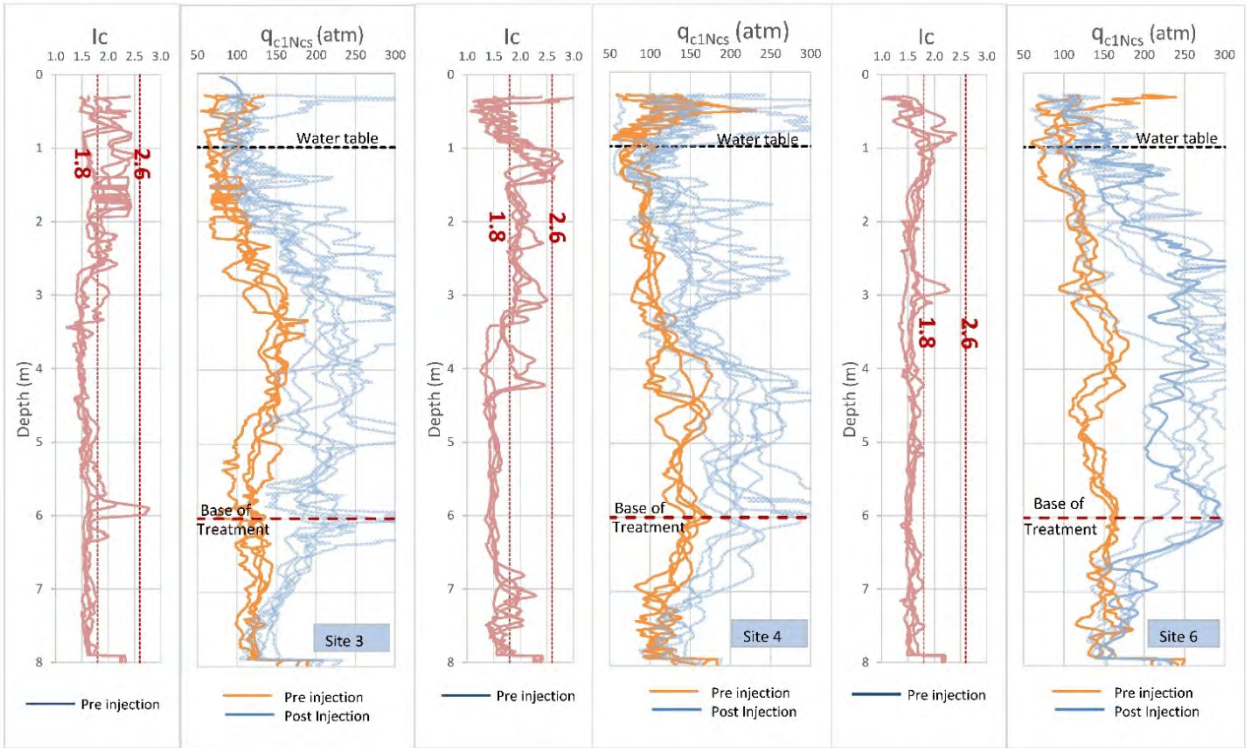


Figure 12 Pre- and post- injection qc1Ncs data

Closer inspection of the results indicates that:

- Soil layers with higher Ic values (greater than about 2.0) have smaller improvements in qc and Vs
- For the same soil types, the percentage of improvement increases with increasing confining pressure – i.e. increased depth and/or surface load.
- For the CPTs closer to the 17kN/m strip loading there is a greater improvement in qc compared to the CPTs further away.
- There is noticeable and consistent improvement in soils for about a metre below the base of the treatment zone.

The apparent fines content (F.C.) of a soil that is inferred from CPT cone data is affected by the densification of a soil when the improvement method also results in increased horizontal stresses. Increases in horizontal stresses result in decreased assessed I_c values, and thus decreased apparent F.C. (The effect on I_c can be seen clearly in Figures 13 to 15.) This in turn artificially lowers apparent q_{c1Ncs} values. When interpreting post-improvement CPT data for q_{c1Ncs}, I_c values were used from the pre-improvement CPT data. This was done by selecting for each post-improvement CPT, the nearest pre-improvement CPT I_c trace that best matched the pattern of the actual post-improvement CPT data as recommended by Nguyen et al. (2014).

Shear wave velocity increased in the order of 35 to 50% on average (about 50 to 75 m/s increases at sites 3 and 4, and 75 to 100 m/s at site 6).

Shear modulus is related to V_s as follows:

$$G = \rho V_s^2$$

Therefore (even assuming a constant soil density) the shear modulus (i.e. the shear stiffness of the soil) has approximately doubled, assuming an average increase of 40% in V_s.

The DMT M_{DMT} value increased on average 130 to 220% (about 50 to 100 MPa increases at sites 3 and 4, and 100 to 200 MPa at site 6).

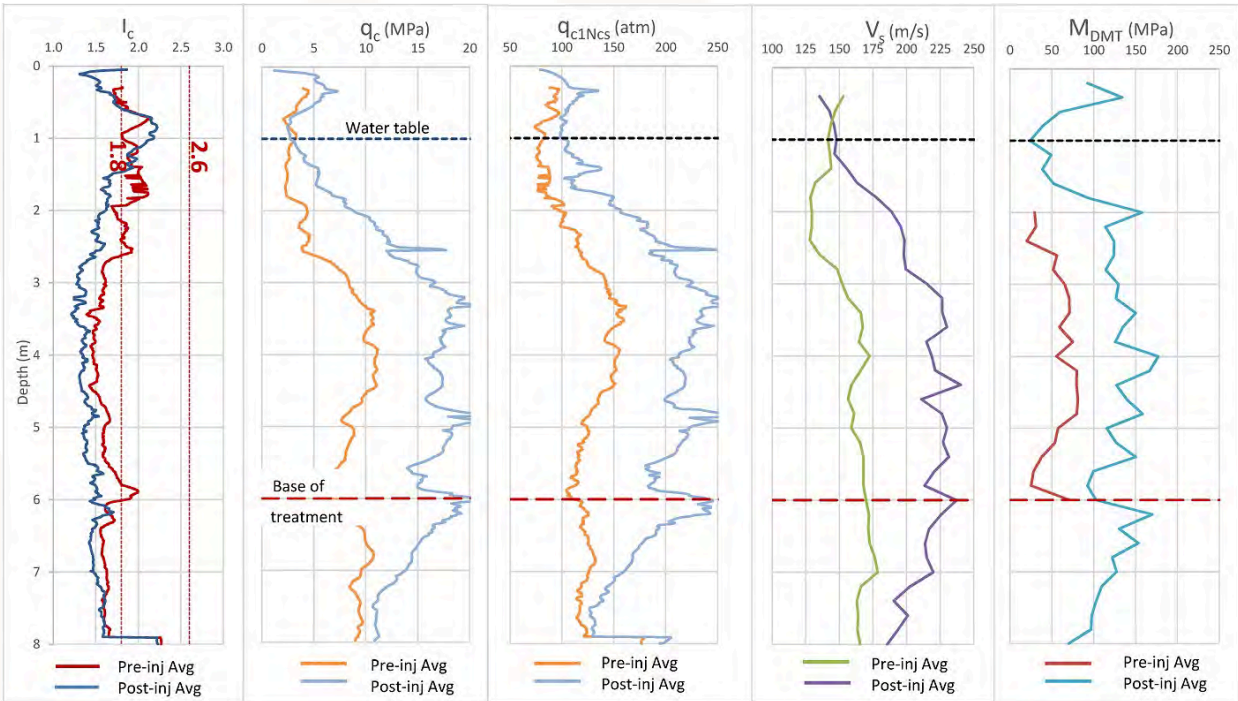


Figure 13 Average of all results Site 3

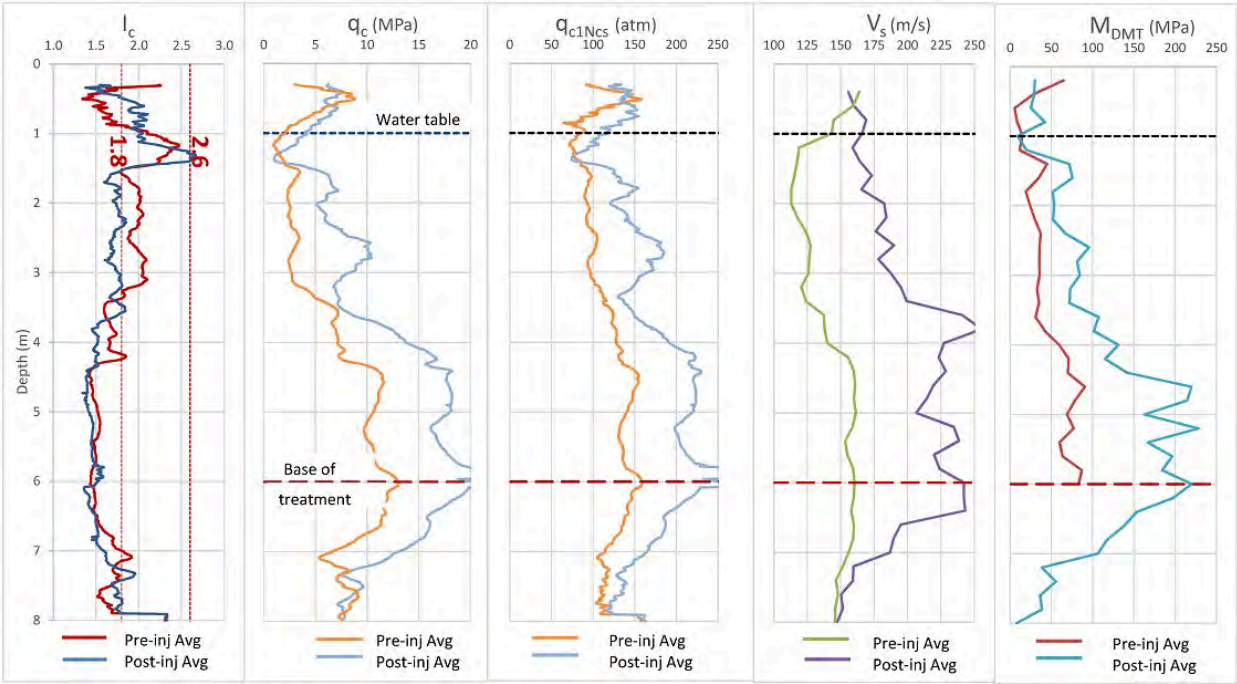


Figure 14 Average of all results Site 4

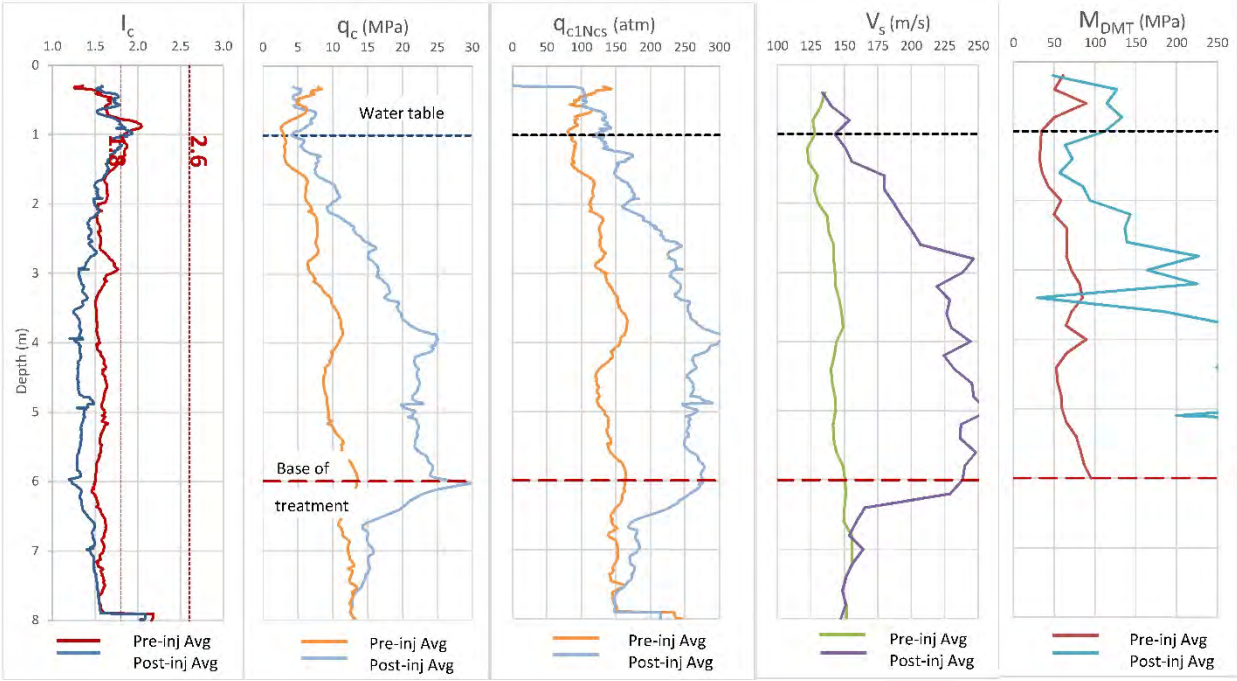


Figure 15 Average of all results Site 6

9.2 Site-Specific Fines Content Correction Factor (C_{FC})

The q_{c1Ncs} data discussed above, as well as the assessments of liquefaction-induced free-field settlements and *Liquefaction Severity Number (LSN)* discussed in the next section, are based on a fines content correction factor (C_{FC}) of 0.2. The C_{FC} parameter is used for adjusting the CPT-derived estimate of fines content based on I_c as proposed by Boulanger and Idriss (2014) - see Figure 16 below. Representative soil samples were retrieved from the borehole drilled at each site, and tested in the laboratory. The fines content results (% passing the 75 micron sieve) indicated that a C_{FC} of 0.2 is appropriate for these sites, as shown in Figure 17.

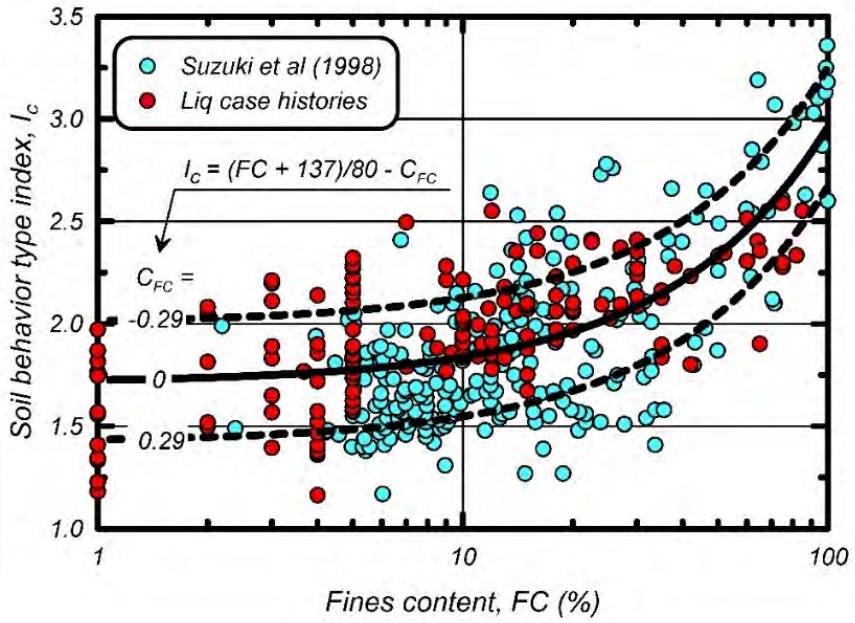


Figure 16 I_c vs FC - Boulanger & Idriss (2014)

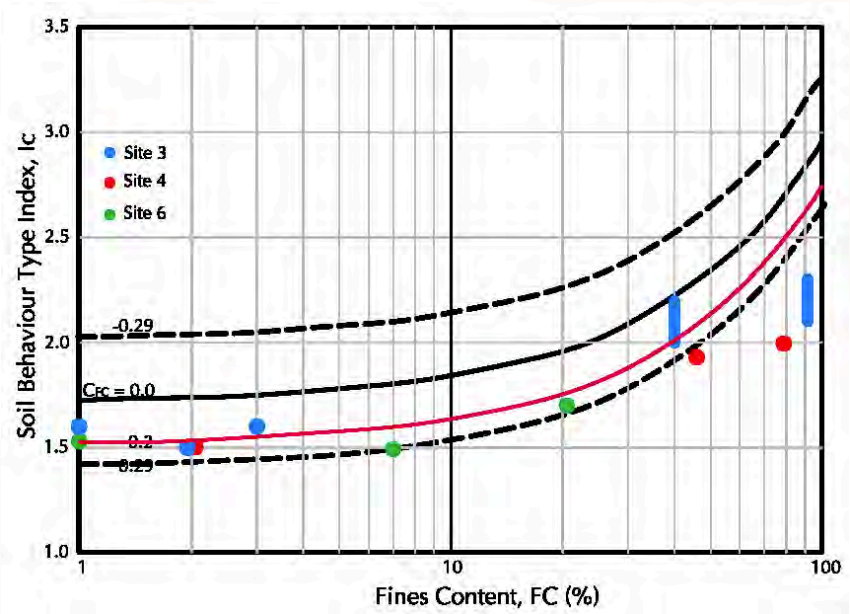


Figure 17 I_C – FC plot used for the three trial sites

9.3 Calculated Post-Liquefaction Settlements

For each CPT the pre- and post-improvement liquefaction triggering potential was assessed using the simplified procedures of Boulanger and Idriss (2014), and corresponding ground surface settlements were computed using Zhang et al (2002). Free-field post-liquefaction consolidation settlements were calculated to 8m depth (where the CPTs terminated) at magnitude M7.5 for a range of PGA values. Settlements were calculated only where data was available for the full depth (i.e. where CPTs had refused early, that data was not included). Not surprisingly, given the increase in post-improvement q_c values, considerable reductions in ground surface settlements are indicated.

Plots of computed pre- and post-improvement free-field ground surface settlement vs PGA are shown in Figures 18 through 20. ('Free field' settlement relates to post-liquefaction volumetric

consolidation, without the possible effects of ejecta being considered).

As the plots below relate only to the soils to a depth of 2 metres below the treated zones, the percentage reduction shown in these plots would be less if a deeper soil profile was considered – conversely, if only the soils in the treated zone (the extent of which can be varied by design) were examined, the percentage increase would be larger.

To illustrate the free-field settlement reductions within a nominal building design framework, each plot is annotated with the 25, 100 and 500 year return period PGAs commonly used for Christchurch. These design levels of shaking correspond to the *serviceability limit state (SLS) and ultimate limit state (ULS)* design levels of shaking, as well as an *intermediate limit state (ILS)* corresponding in this instance to a chosen return period of 100 years. The plots show significant reductions in the computed free field settlements in each of these design cases, and in fact across the full spectrum of PGAs examined.

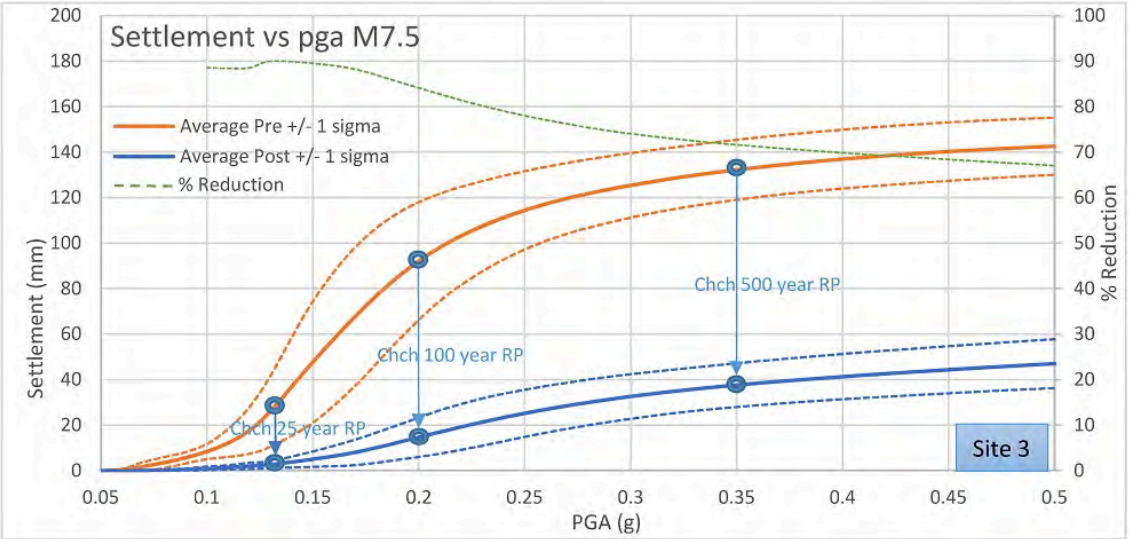


Figure18 Site 3 Computed free-field liquefaction settlement response with PGA

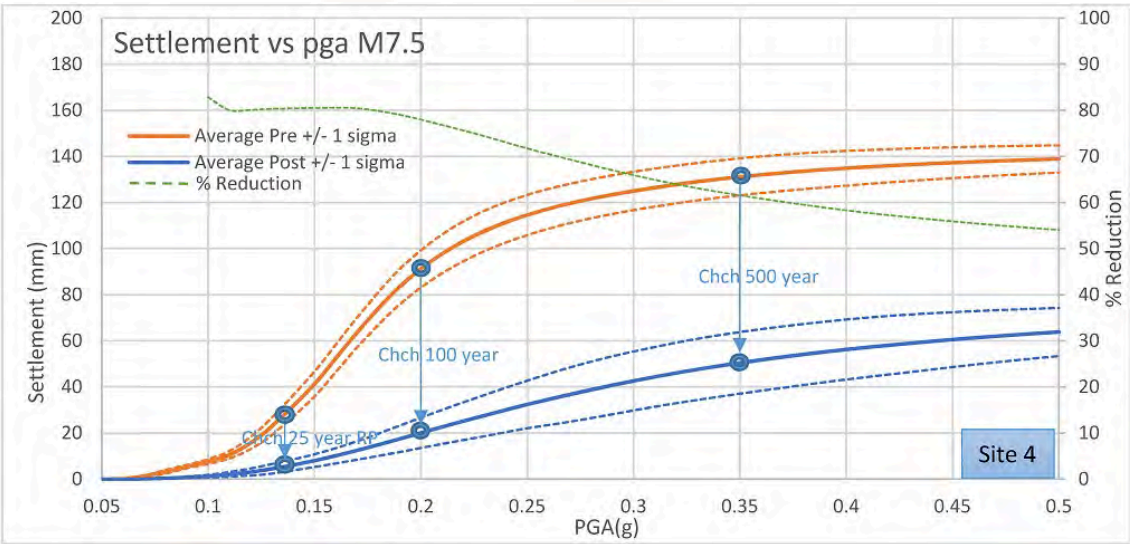


Figure 19 Site 4 Computed free-field liquefaction settlement response with PGA

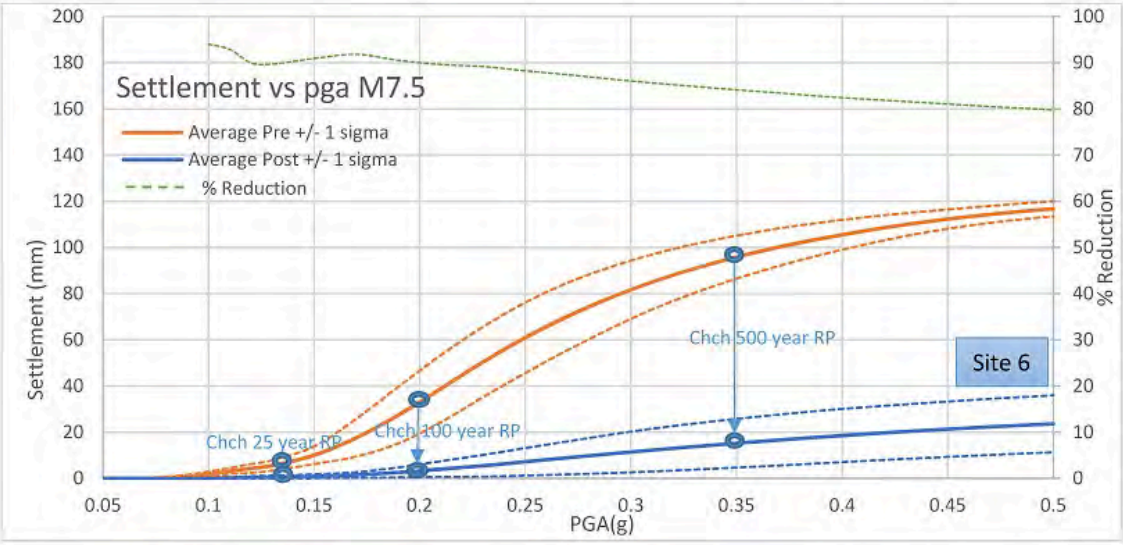


Figure 20 Site 6 Computed free-field liquefaction settlement response with PGA

9.4 Calculated Post Liquefaction Liquefaction Severity Number (LSN)

The Liquefaction Severity Number (LSN) is a depth-weighted index proposed by van Ballegooy et al (2014) to give an indication of the possible degree of surface land damage that might result from liquefaction. For each test panel, post-liquefaction LSN values were calculated for a range of PGAs (at a magnitude of M7.5). As with calculated settlements, LSN values were calculated only where data was available for the full depth treated. As shown in Figures 21 – 23, there is a clear trend of a reduction in LSN at all three sites.

[It should be noted that CPT data was only collected to a depth of 8 metres for most of the investigation points. LSN is however calculated over a depth of 10 metres, therefore the assessed values in this study are not 'true' LSN numbers. However, the LSN parameter weights settlement data in inverse proportion to depth, and therefore the lower 2 metres of the soil profile in fact contribute comparatively little to the calculation. In order to illustrate this effect, one of the post-improvement CPTs at site 6 was extended to 10m depth, and a comparison of LSN was carried out using both 8 metres of data and 10 metres. The differences were very small, a LSN difference of 0 - 1 up to a PGA of 0.4g (M7.5) and 2 at a PGA of 0.5g.]

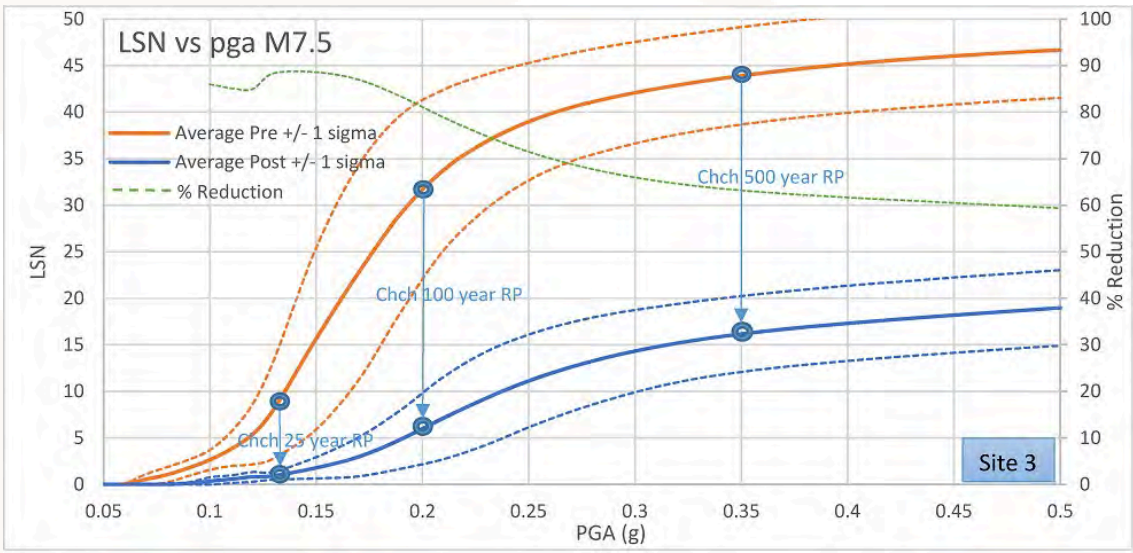


Figure 21 Site 3 Response of LSN with PGA

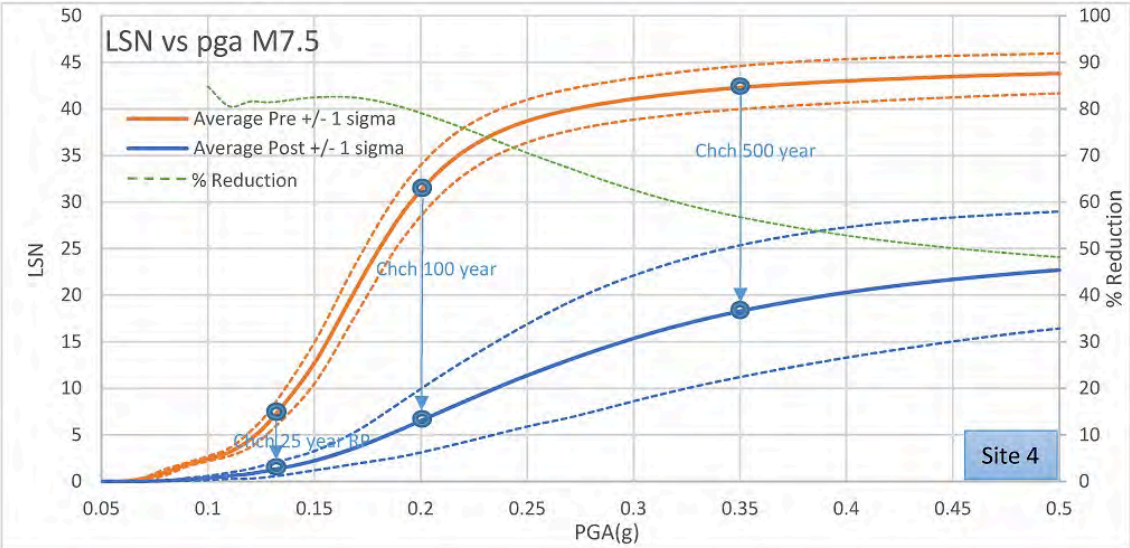


Figure 22 Site 4 Response of LSN with PGA

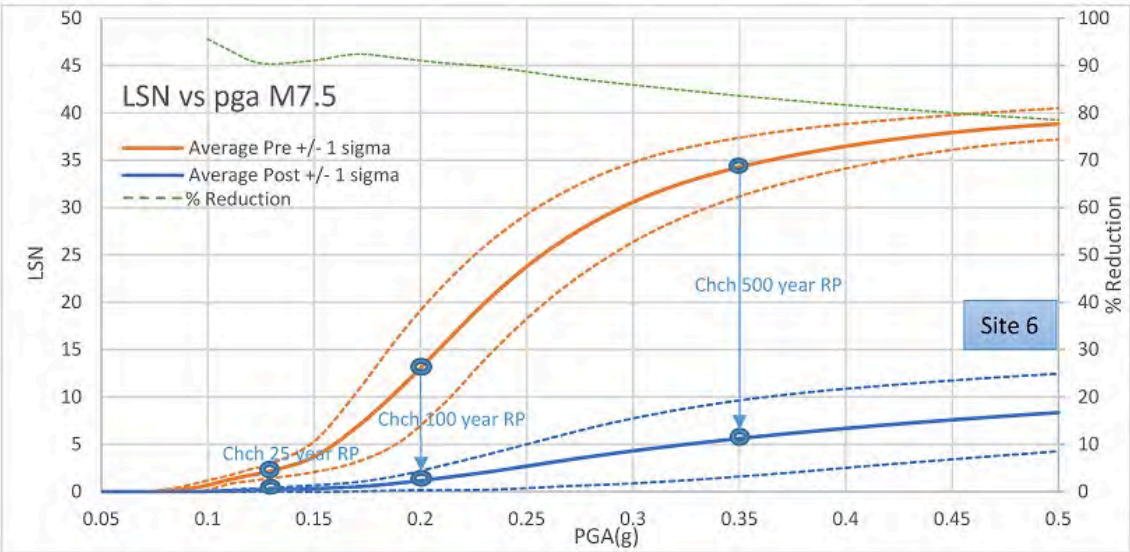


Figure 23 Site 6 Response of LSN with PGA

9.5 Horizontal Stress Effects

The DMT tests show an increase in soil stiffness based on DMT modulus, as well as increases in the derived 'horizontal stress index' parameter, K_D which increased on average across all three sites from $K_{D,\,ave}$ = 7 pre-improvement to $K_{D,\,ave}$ = 13 post-improvement. This implies an increase in the at rest lateral earth pressure coefficient, K_0.

Marchetti et al (2001) relate K_D to at rest lateral earth pressure (K_0) as follows:

$$K_0 = 0.376 + 0.095K_D - A\frac{q_c}{\sigma'_{v0}} \qquad (1)$$

Where: K_o = at rest lateral earth pressure

K_D = DMT horizontal stress index

A = 0.002 for 'freshly deposited' sands

q_c = measured CPT cone tip resistance

σ'_{v0}= effective overburden stress

It is generally accepted that the liquefaction resistance of a soil increases with increasing K_0 (Ishihara et al. 1977, Seed 1979). However, it is often assumed that correlations between measured in-situ parameters such as CPT q_c and the soil's resistance to liquefaction (i.e. cyclic resistance ratio CRR) is relatively independent, on the basis that an increase in K_0 will produce a comparable increase in CPT q_c, or SPT 'N'(Seed 1979).

Salgado et al (1997) however, found that "*CPT liquefaction correlations appear to be reasonably independent of K_0 for normalized tip resistances q_{c1} less than about 12 MPa. However, when K_0 exceeds about 1.2 times its normally consolidated value …. neglecting the effects of K_0 on the CPT liquefaction correlations can be significantly unconservative for q_{c1} values greater than about 12 MPa. The proposed procedure may be of practical significance when evaluating ground improvement techniques that derive some of their observed benefit from increases in the in-situ lateral stresses*".

Furthermore, Harada et al (2010) also found enhancements in CRR from lateral stress effects over and above those derived only from CPT data, but reports that the effects are greater for looser deposits than they are for denser materials: "*generally the resistance to liquefaction increases with increasing K_C value, little difference was noted when the density of the deposit was high".*

From this it is apparent that the increases in site performance that are discussed in the preceding Section 9.3 and 9.4 are likely to be somewhat conservative – however the degree of conservatism appears to be the subject of ongoing discussion in the research community.

There is also some debate as to the longevity of horizontal stress effects from ground improvement. For this reason, and the ongoing discussion regarding the degree of potential enhancement, these effects have not been included in any of the forgoing analyses (in Section 9.3 and 9.4) of the degree of ground improvement achieved by the resin injection, nor in the reported improvements in the summary of Section 10 of this report.

The DMT testing was carried out some 2 weeks after the improvement works were installed. It would be informative to revisit the site approximately 6 months after these first tests were carried out, to retest the site and see if there is a significant reduction in this effect with time.

9.6 Static Bearing Capacity and Settlement.

Although the primary focus of this study was ground improvement for liquefaction mitigation, densification of soils will also lead to enhanced static bearing capacities and settlement characteristics for shallow foundations.

Pre-improvement and post-improvement shallow foundation bearing capacities were assessed from the CPT data for Site 3 as a preliminary assessment of this effect (Schmertmann, 1978), assuming a nominal founding depth of 0.5m. The results (refer to Figure 24) indicate bearing capacity increases of approximately 20% to 75%. The increases are more noticeable at wider footing dimensions, as these footings are more influenced by the strength of the deeper soil layers (which is where the ground improvement is more effective due to increasing confining pressures.)

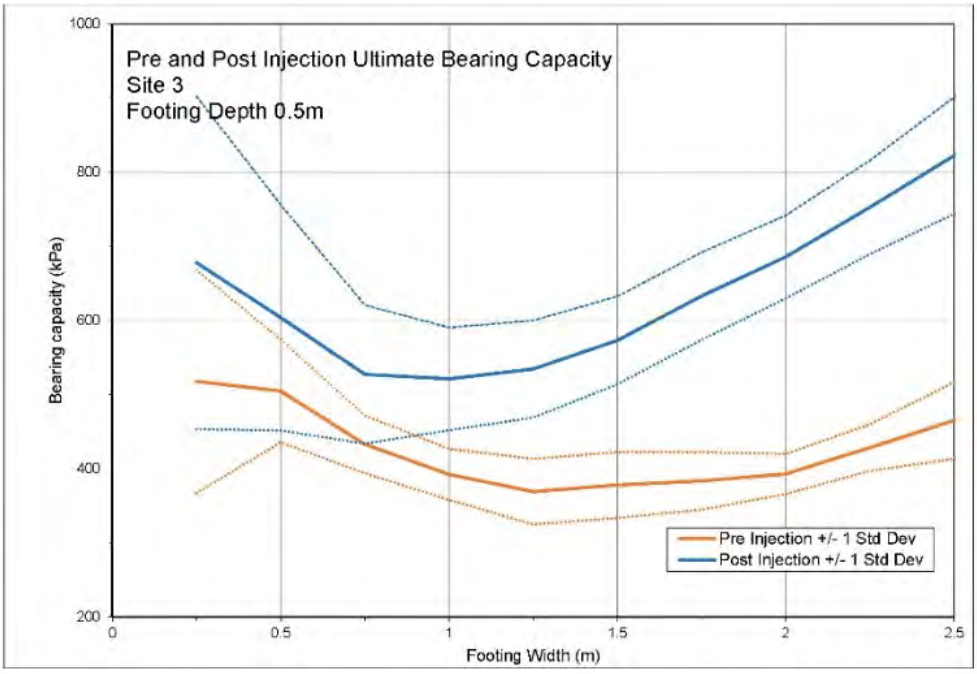


Figure 24 Pre- and post-improvement bearing capacity (strip footing) Site 3

Pre-improvement and post-improvement shallow foundation settlements were also assessed from the CPT data for Site 3 (Schmertmann, 1970). The analysis was carried out to estimate the foundation contact pressures required to limit settlements to 25mm.. The results (refer to Figure 25) indicate increases in contact pressures of approximately 15% to 60%. As for bearing capacity, the increases are more noticeable at wider footing dimensions.

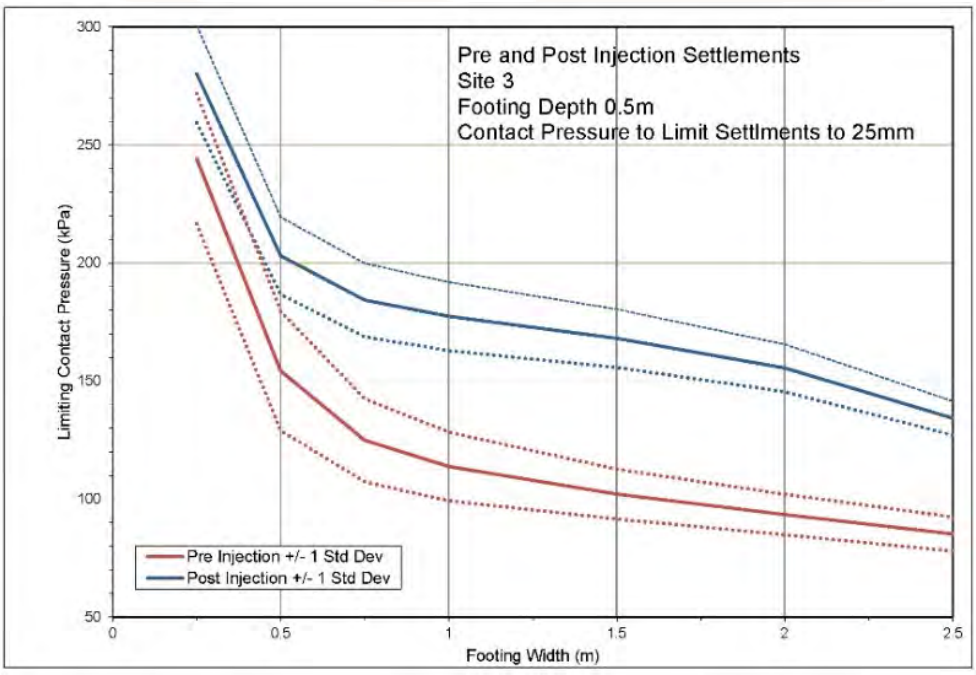


Figure 25 Pre- and post-improvement settlements (strip footing) Site 3

9.7 Modulus of Subgrade Reaction

A series of plate load tests (PLT) were carried out at each site. This was done after the completion of the main trials, therefore the 'pre-injection' testing was carried out on untreated ground just

outside each of the treated panels. Post-injection testing was performed within each test panel. The purpose of the testing was to assess the effects of the resin injection on near-surface soil stiffness (i.e. Modulus of Subgrade Reaction). It is noted that the resin injection trials were not specifically designed to enhance modulus of subgrade reaction.

A steel plate of 610mm diameter was used at a depth of 0.5m below ground level for the testing, loaded in stepped increments up to a maximum of 350 kPa.

The results show a stiffening of the soil response at each of the sites, as illustrated in Figure 27 below. The initial stiffness of the soils at each site (i.e. at lower levels of load) were similar, and not significantly affected by the soil treatment. At higher levels of loading however, Sites 3 and 4 showed a softer soil response and also a good response to ground treatment by resin injection.

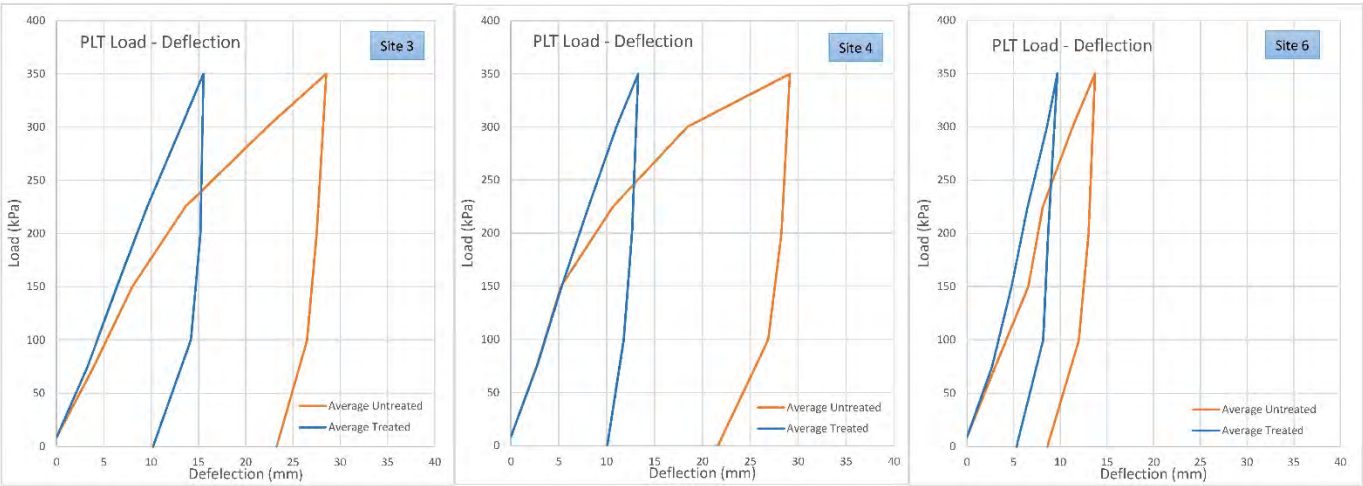


Figure 26 Plate Load Test Load-Deflection Curves: Site 3, 4 and 6

Modulus of Subgrade Reaction was assessed for each test between the 3rd and 4th loading cycle (225 to 300 kPa), and increases of 50% - 90% were noted between the treated and untreated ground. (see Table 5 below).

Table 5 Modulus of Subgrade Reaction Test Results

Site	Modulus of Subgrade Reaction ($MN/m^2/m$)				% Improvement
	Untreated ground		Treated Ground		
	Range	Average	Range	Average	
3	12 - 16	14	20 - 26	22	57%
4	12 - 17	15	24 - 31	28	90%
6	23 - 25	24	30 - 43	37	52%

It is concluded therefore that foundation soil stiffness can be successfully improved using resin injection ground improvement.

9.8 Time Effects

To assess whether the observed improvement in soil density changed with time (based on CPT q_c data), a round of additional CPT testing was undertaken approximately 90 days after the conclusion of the injection process. As can be seen in in Figure 28, the CPT data at 90 days generally falls within the scatter of data collected two weeks after the injection process was carried out. It is noted in the results that the CPT plots that fall at the lower end of the clatter were those located furthest from the foundation line load surcharge. It is also noted that both

this trial and the 2013 EQC trials demonstrated that CPT data is highly variable at these sites (even across distances of only a metre or two). It is concluded therefore that it is unlikely there has been a statistically significant change in CPT cone resistance with time.

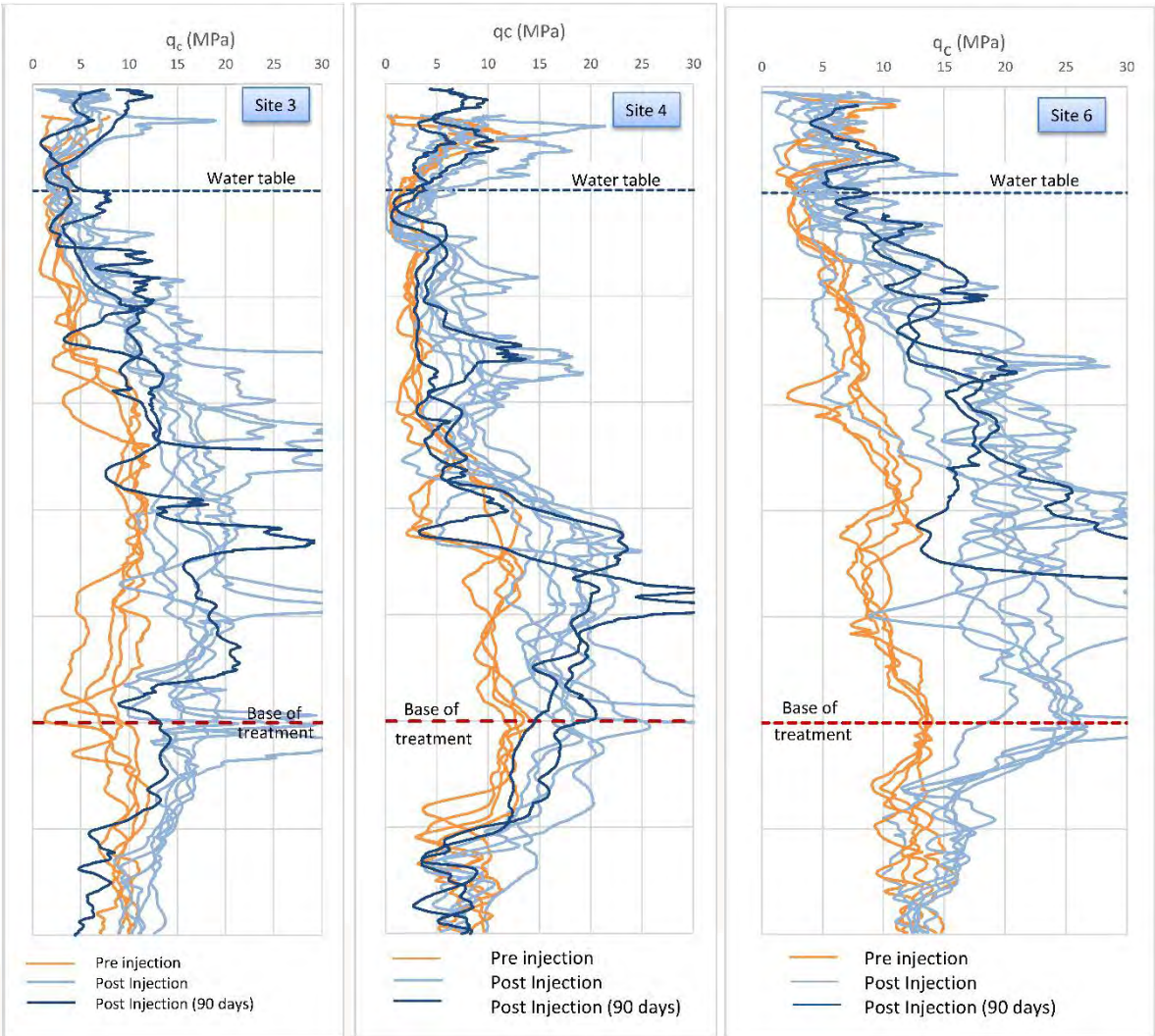


Figure 27 Ninety-Day CPT results: Sites 3, 4 and 6

10 Summary and Discussion of Results

As can be seen the preceding results and analyses, resin injection has demonstrably improved the density and stiffness of the treated ground, as measured by several insitu testing methods.

CPT cone resistance (q_c) has increased noticeably - in the order of 80% on average for these sites and surface loads - and therefore it is implied that soil densities have significantly increased. The derived value q_{c1Ncs}, which accounts for soils fines content, has also therefore increased considerably (on average 70%). This variable (q_{c1Ncs}) is used in the calculation of liquefaction triggering and therefore also settlements and LSN values. A 50% to 80% reduction in calculated settlements was achieved at all three test sites, and LSN values have decreased by similar amounts (Sections 9.3 and 9.4). The implied surface damage potential for these sites from seismic liquefaction has therefore reduced significantly.

Measured post-improvement cross-hole V_s values have increased in the order of 40%, indicating a significant increase in soil stiffness, with the shear modulus (i.e. the shear stiffness of the soil) approximately doubling.

Post-improvement DMT has indicated increases in K_D of about 85%, which (as discussed in Section 9.5) implies a potential further increase in CRR beyond that assessed only from increases in q_{c1Ncs}. (Due to current uncertainty over the magnitude and the longevity of this effect, this has not however been taken into account in the previously discussed settlement or LSN results).

The primary focus of the study was liquefaction mitigation, and therefore the resin injection was targeted at soils below the water table (i.e. not the shallower surface materials, which typically have more effect on static bearing capacities for shallow foundations). However, the post-improvement test data shows that bearing capacities for shallow foundations has been improved at these sites, and potential static settlement decreased (Section 9.6). This suggests that with a different injection strategy, this methodology can be applied to buildings where there is a need to increase bearing capacities, or reduce settlements of shallow foundations. This might, for example, be useful where it is intended to increase loadings on an existing building, through a change of use or through the construction of additional stories.

A qualitative assessment of the data indicates that increasing fines content of the soils leads to less of an increase in improvement in response to resin injection (as is typical of other improvement methods that rely primarily on densification of the soil). This can be seen in the greater degree of improvement at Site 6, which is a predominantly sandy site, than at sites 3 and 4 (which have a number of siltier layers). It can also be discerned when examining the way the measured parameters vary with I_C at the individual sites – although when examining Figures 13 and 14 (for example) this effect appears to be also mixed with a confining stress effect, where the layers with less of an increase in CPT q_c are siltier but also shallower.

The fines content effect is more evident when examining the LSN vs PGA plots of Figures 21 – 23. LSN tends to emphasise the performance of soil layers nearer to the ground surface. At sites 3 and 4 there is a silty near-surface layer, whereas at site 6 this layer is absent, hence the LSN response at Site 6 is greater. However, the increases in measure parameters at Sites 3 and 4 are still significant.

Meaningful increases in CPT q_c were observed up to at least an I_C value of 2.0. It is unfortunate that at these sites there were very few soil layers with I_C values in the range 2.2 to 2.6; particularly at depth, to allow a more refined assessment of the upper limit of fines content for 'treatable' soils. It would be useful to further examine the data from this trial, as well as from production installations, to explore whether the increase in soil strength parameters can be

better quantified with respect to influencing properties such as soils fines content (e.g. I_C) and confining stress.

Increasing confining stress (either by way of depth in the soil profile or the application of a surface surcharge) does appear to result in a better response to the treatment. On sites where treatment is required for a building that is lightweight in nature, surcharging can often be practically applied using a system of portable weights – for example water-filled IBCs (Intermediate Bulk Container – a pallet water tank), concrete blocks or lead weights can all be employed.

11 Conclusions

The results of this study demonstrates that resin injection is a viable ground improvement method for mitigation of liquefaction potential, and also for increasing foundation bearing capacities in sandy soils (including siltier sands – e.g., sandy soils with CPT I_C values up to about 2.0). Significant improvements in soil density, stiffness and strength have been achieved as demonstrated by in-situ testing.

It has been noted that decreasing fines content, and increasing confining pressures, lead to better densification effects in treated soils. While the fines content of a soil deposit may constrain the applicability of this technology to any particular site, confining pressures can be applied through the use of portable kentledge on a site.

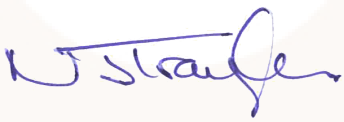
Ground heave can occur when injecting close to the ground surface, but this depends on the degree of improvement being targeted (and therefore the amount of resin material being injected), soil conditions, and the type of resin materials that are used. If this does occur, final building levels can be corrected with further controlled injection processes. In cases where a building also requires level correction, some controlled ground heave can be beneficial. In other cases allowances will need to be made so that the building can accommodate some changes in final floor level. For heavy buildings, or on sites where the soils are only being treated below 2m depth, significant ground heave may not occur at all.

This trial, and also previous commercial application has shown that resin injection methodology can be successfully applied not only to cleared sites, but more importantly, to ground beneath existing buildings, structures, and critical infrastructure.

In appropriate soil conditions, ground improvement method by resin injection is useful for both liquefaction mitigation in seismically active areas, and for general bearing capacity enhancement under static conditions.

Yours faithfully,

Geotech Consulting Ltd per:



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12 References

Boulanger, R.W. & Idriss, I.M. 2014. CPT and SPT based Liquefaction Triggering Procedures. *Report No UCD/CGM-14/01 Dept of Civil & Environmental Engineering, College of Engineering, University of California at Davis.*

Erdemgil, M., Saglam, S., Bakir, B. 2007. Utilization of Highly Expansive Polymer Injection to Mitigate Seismic Foundation Failure for Existing Structures. *Proc. 8th Pacific Conference on Earthquake Engineering.*

EQC (in prep). Ground Improvement Trials Report.

Harada, K., Orense, R., Ishihara, K., Mukai, J., 2010. Lateral Stress Effects on Liquefaction Resistance Correlations. *Bulletin of the NZ Society for Earthquake Engineering, Vol 43, No.1, March 2010.*

Ishihara, K., 1977. Simple Method of Analysis for Liquefaction of Sand Deposits During Earthquakes. *Soils and Foundations, Vol. 17, No. 3, pp. 1-17.*

Jamiolkowski, M., LoPresti, D., Manassero, M., 2001. Evaluation of Relative Density and Shear Strength of Sands from Cone Penetration Test and Flat Dilatometer Test. *Soil Behaviour and Soft Ground Construction (GSP 119), American Society of Civil Engineers, Reston, Va., 2001, pp. 201–238.*

Marchetti S., Monaco P., Totani G. & Calabrese M. 2001. The Flat Dilatometer Test (DMT) in Soil Investigations. Report by the ISSMGE Committee *TC16. Proc. IN SITU 2001, Intnl. Conf. On In situ Measurement of Soil Properties.*

Nguyen, T. et al. 2014. Proposed Modification to CPT-Based Liquefaction Methodology for Post-Vibratory Ground Improvement. *Geo-Congress 2014*.

Robertson, P.K. and Wride, C.E. 1997. Cyclic Liquefaction and its evaluation based on SPT and CPT. *Proceedings, NCEER Workshop on Evaluation of Liquefaction Resistance of Soils.*

Salgado, R., Boulanger, R. W., and Mitchell, J. K. 1997. Lateral stress effects on CPT liquefaction resistance correlations. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 123(8), 726-735.*

Schmertmann, J.H. 1970. Static Cone to Compute Static Settlement Over Sand. *Journal of Soil Mechanics and Foundation Design, ASCE, vol 96, SM3.*

Schmertmann, J.H. 1978. Guidelines for Cone Penetration Test: Performance and Design. *FHWA-TS-78-209 report, US Dept of Transportation.*

Seed, H. B., 1979. Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes. *Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT2, pp. 201-255.*

Traylen, N., van Ballegooy, S., Wentz, R. 2016. Liquefaction Mitigation beneath Existing Structures Using Polyurethane Grout Injection. *NZSEE Conference 1-3 April 2016 Christchurch NZ.*

van Ballegooy, S.; Cox, S. C.; Thurlow, C.; Rutter, H. K.; Reynolds, T.; Harrington, G.; Fraser, J.; Smith, T. (2014) Median water table elevation in Christchurch and surrounding area after the 4 September 2010 Darfield Earthquake: Version 2, *GNS Science Report 2014/18, April 2014.*

van Ballegooy, S., Malan, P., Lacrosse, V., Jacka, M., Cubrinovski, M., Bray, J. D., O'Rourke, T. D., Crawford S. A., Cowan, H. 2014. Assessment of liquefaction-induced land damage for residential Christchurch. *Earthquake Spectra, February 2014, 30 (1), 31-55.*

Wotherspoon, L.M., Cox, B.R., Stokoe, K.H., Ashfield D.J., Phillips, R.A. (2016) Utilising direct-push crosshole testing to assess the effectiveness of soil stiffening caused by installation of stone columns and Rammed Aggregate Piers. *6th International Conference on Earthquake Geotechnical Engineering 1-4 November 2015 Christchurch NZ.*

Zhang, G., Robertson, P., & Brachman, R. 2002. Estimating liquefaction-induced ground settlements from CPT for level ground. *Canadian Geotechnical Journal, 39(5), 1168-1180*

Attachment 1

EQC Ground Improvement trials, 2013

A series of trials were undertaken in Christchurch in the latter half of 2013, to examine the performance of various forms of ground improvement. The trials were carried out on behalf of EQC, with support from MBIE as well as the National Science Foundation.

A number of ground improvement methods were constructed on residential land that had been abandoned en-masse following the C.E.S. ('Red Zone' land). These methods were 'cut down' from normal treatment depths to see if, for residential use, they could achieve acceptable outcomes at lower cost than methods currently in use at the time. The tested methods included:

- Rammed aggregate piers ("RAP").
- Rapid impact compaction ("RIC").
- Driven timber piles.
- Low mobility grout columns ("LMG").
- Compacted and reinforced gravel raft.
- Compacted and cement stabilised sand raft.
- **Resin injection.**
- Horizontally soil mixed beams.

The installed ground improvement works were tested in a variety of ways. Generally both Cone Penetration testing ("CPT") and direct push cross-hole testing ("Vs/Vp") was carried out (see Figure 28) (both pre and post installation) where appropriate.

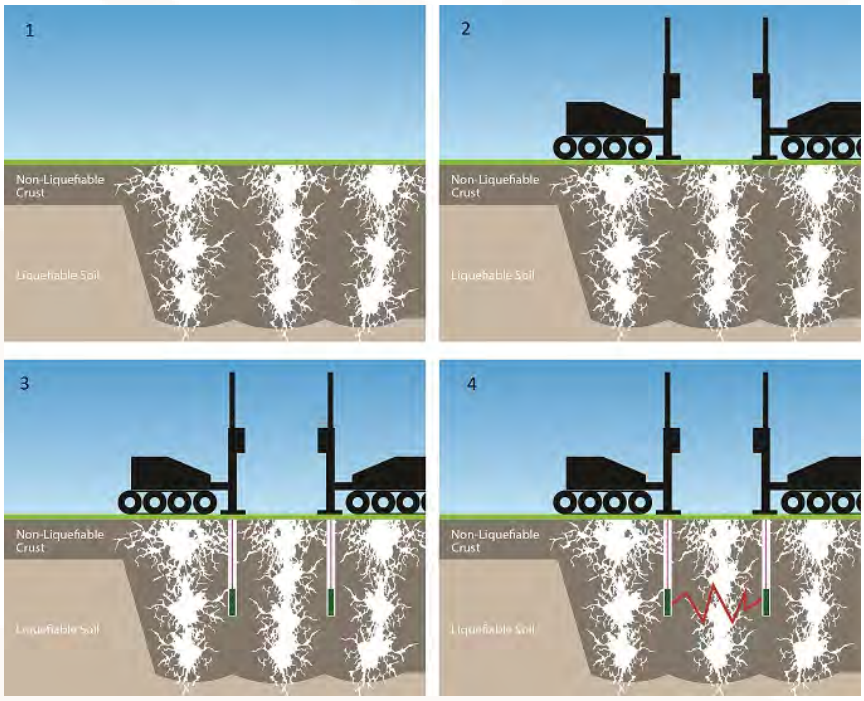


Figure 28 Direct push cross-hole (Vs/Vp) testing

Most of the methods were also tested by shaking the ground with controlled input motions with a large vibroseis machine (see Figure 29 - this is the 'T-Rex' machine from the University

of Texas). With this machine a large weighted plate is placed on the ground surface above the installed ground improvements, and shaken horizontally to impart simulated 'shaking' into the ground. Pore water pressure response was measured during the T-Rex shaking, to give an understanding of how close to liquefaction (i.e. when the pore water pressure equals the soil overburden pressure) each method came when compared to unimproved ground.

Figure 29 University of Texas T-Rex vibroseis machine

Finally, large scale vibrational motions were imparted to the ground via controlled explosions, in order to liquefy the underlying soils and therefore to see how the improved layers performed in terms of measured ground deformations during a liquefaction event (Figure 30).

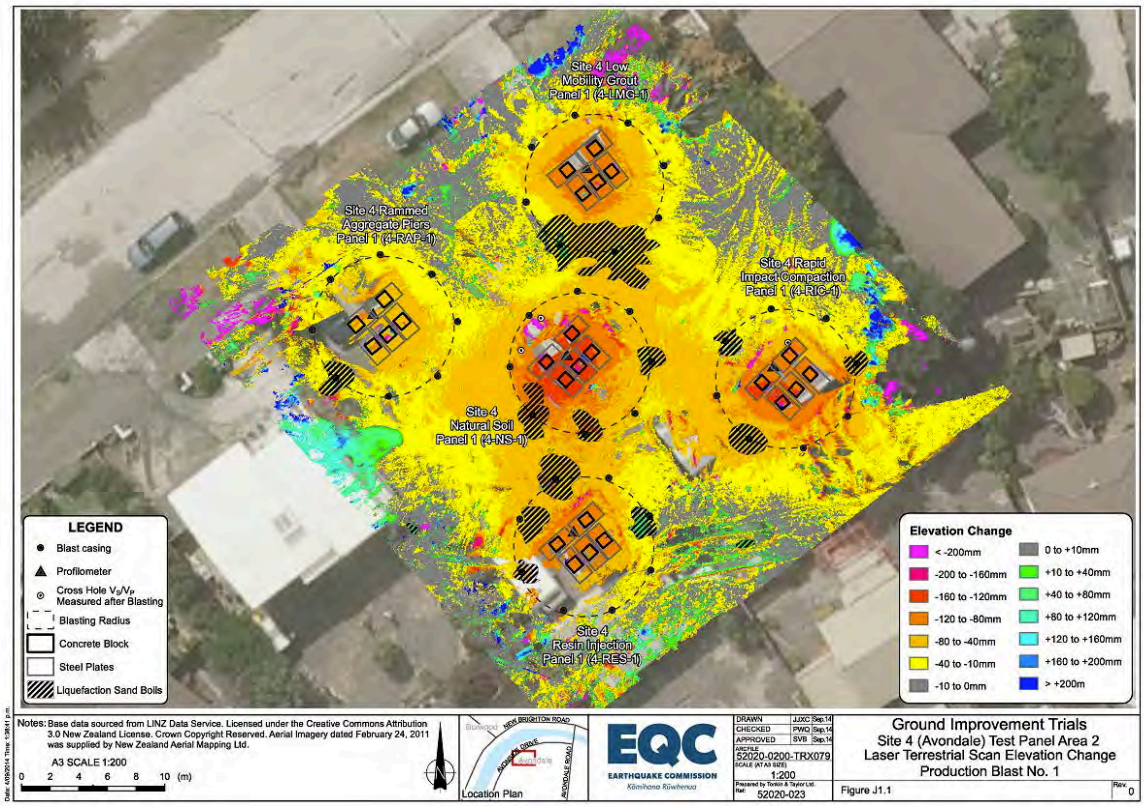


Figure 30 Lidar ground settlement data from blast trials

EQC Trial Results

Resin injection was included (at a late stage) in the 2013 EQC trials, to see if this method

(when used for level correction) might degrade the liquefaction performance of the ground (as had been found for LMG). This was found not to be the case, and in fact an improvement in the liquefaction performance was noted. The preliminary examination of this technology indicated two main potential improvement mechanisms. CPT testing generally showed that the soil densities appeared to increase, and direct-push cross-hole geophysical testing showed an increase in the composite shear stiffness of soils that had been treated in this manner Additionally (from geophysical P-wave test results) an apparent desaturation effect was noted in the soils. The mechanism of the apparent desaturation is at yet undetermined.

CPT cone tip resistance increased up to 5 MPa over the full height of the improvement zone. (In some layers however there was no significant improvement).

Cross-hole shear wave velocity testing, which tests the stiffness of the composite soil/resin block, showed an increase of shear wave velocity (Vs) from 100 – 150 metres per second (m/s) to 150 – 200 m/s. The improvement in Vs was 15 to 40% over unimproved natural ground.

The T-Rex testing indicated cyclic shear strains (i.e. how much the soil moves in shear with each reversal of the ground movement) in the treated soils of less than half that of the untreated soils.

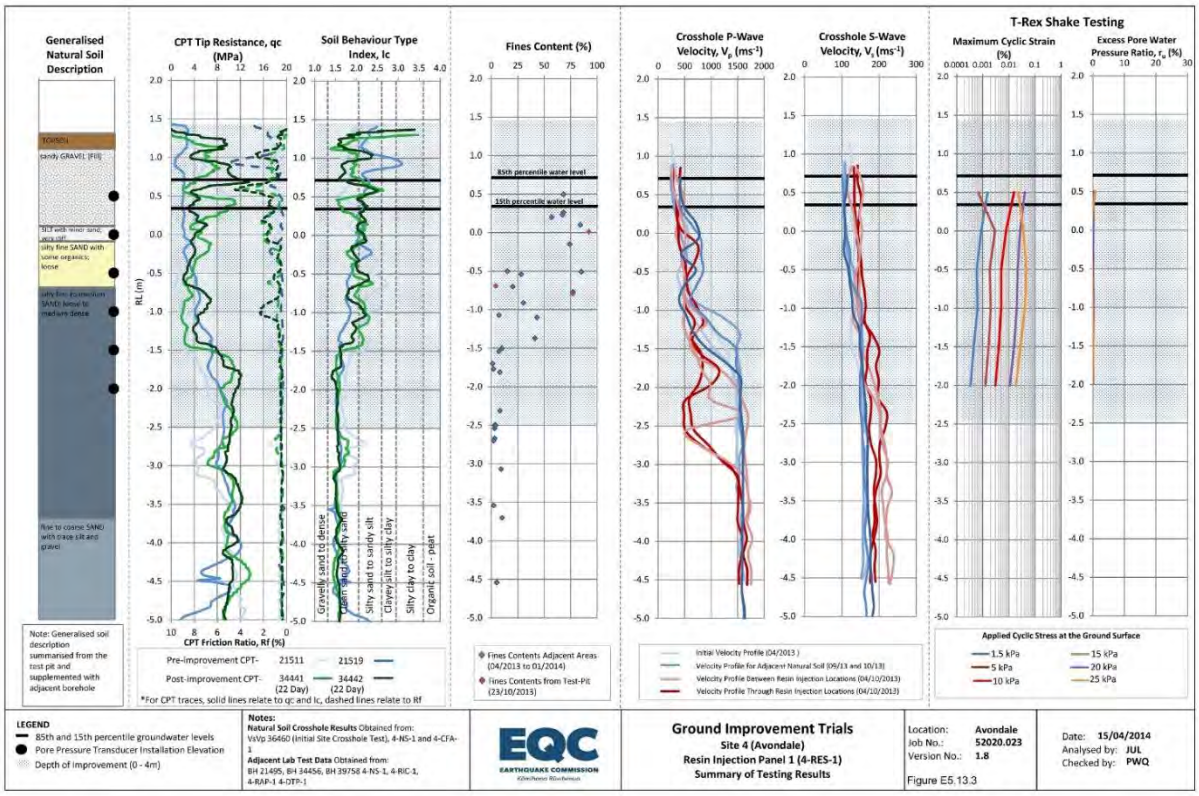


Figure 31 Summary page of tests on resin panel from EQC draft report.

Porewater pressure response during T-Rex testing was also measured. This showed very little increase in porewater pressure during the T-Rex shaking. This would normally be seen as indicating very good liquefaction resistance. However P-wave testing was also carried out at the same time to give an indication of where full soil saturation was occurring in the soil column. The testing showed that the resin injection process appeared to have desaturated the soils, and therefore there could be no porewater pressure response to the shaking (if the

soil was not fully saturated). The longevity of this apparent desaturation is unknown at this point, and therefore this mechanism is currently discounted as a reliable one for liquefaction mitigation, unless longevity of the desaturation could be assured.

Ground settlement measurements taken after blast testing the trial panel (to induce liquefaction in the underlying soils) indicated much less overall settlement and differential settlement when compared with adjacent untreated ground.

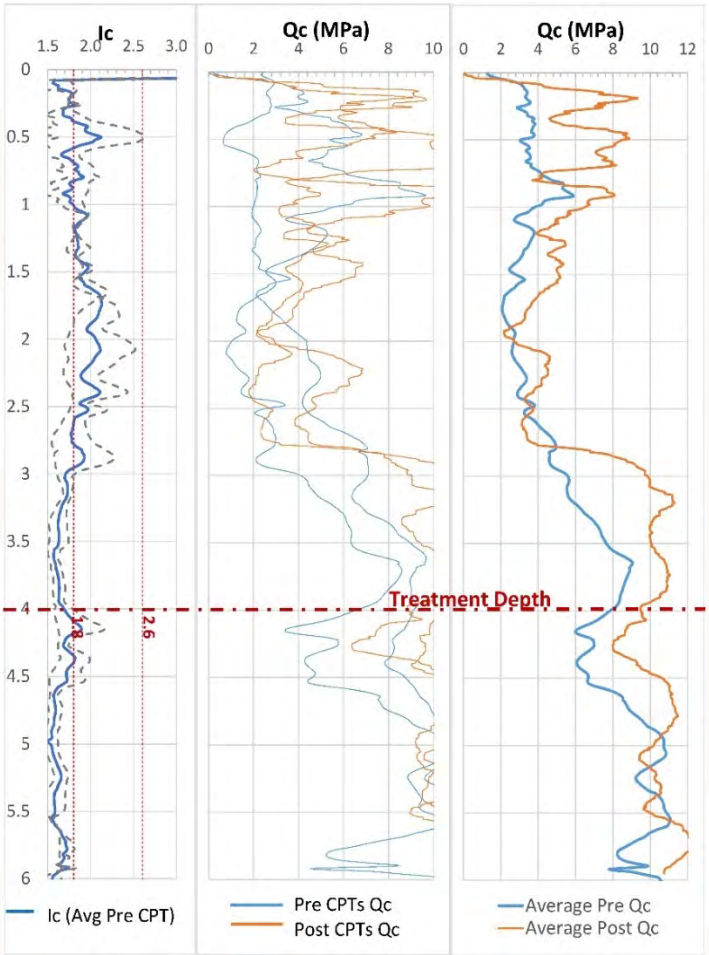


Figure 32 EQC 2013 ground improvement trial resin results

EQC Trial Conclusion

The Resin Injection panel that was tested did show an increase in liquefaction resistance via a number of measures. The overall density of the soil increased, as measured by CPT tip resistance. The composite stiffness of the improved soil block increased, as measured by shear wave velocity testing. The cyclic strains in the soils during shaking were decreased, as measure during T-Rex testing. Pore pressure response during shaking was dramatically decreased, and settlements during blasting trials was also much reduced.

However the decrease in pore pressure response during shaking is for the time being effectively discounted due to the desaturation of the soils caused by the resin process (because of doubts over the longevity of that effect). This may also cast some doubt over the

magnitude of the decrease in measured settlements during the blasting trials.

After testing was complete the trial panel was partly exhumed and it was noted that there was some limited permeation of the resin into the clean sands – the cementation by permeation may also have contributed to the increase in liquefaction resistance.

Despite the indications that resin injection did result in improvement in liquefaction resistance of soils, it is our understanding that the draft EQC report states that it 'cannot be currently concluded that it is in fact an effective mitigation technique'. This is because of the issues outlined above regarding desaturation, but mainly because only one panel was tested and therefore the data set, based on limited test numbers and locations, is regarded as being too small to be conclusive.

However, it was recommended that this technique should be subjected to further study given the promising early results.

Attachment 2

Commercial Shopping Centre

Three adjoining large format retail buildings that suffered liquefaction-related settlement damage in the 2010 -2011 Canterbury Earthquake Sequence (up to 160 mm differential settlement across the 90 metre by 60 metre combined building footprint) were relevelled, repaired and upgraded in late 2015 and early 2106. The relevelling was carried out using Mainmark 'JOG™' (a computer controlled cement based micro-injection process) and Teretek™ resin injection methods, followed by structural strengthening of the buildings.

The buildings in this case were upgraded beyond basic requirements in terms of percentage of new building standard, ("% NBS") with both additional structural strengthening, and liquefaction mitigation by densification and stiffening of the underlying shallower soils (treating variously to 4m or 7m depth) using the resin injection methodology. The objective of the ground improvement works was to reduce liquefaction-induced damaging differential settlements at ULS levels of shaking, as well as to reduce the commercial operational risks for tenants occupying the buildings by providing good performance at SLS levels of shaking.

The liquefaction mitigation works were carried out as a 'Design and Construct' project by Mainmark Ground Engineering (NZ) Ltd, in advance of the other relevelling and strengthening works, using pressure injection of an expanding resin grout mix into the ground at depth, to densify the underlying soils. This was achieved by drilling holes through the concrete floor slabs or surface fill, inserting grout tubes into the ground to the desired depth, and introducing the resin mix at the grout tube tip as the tubes were withdrawn.

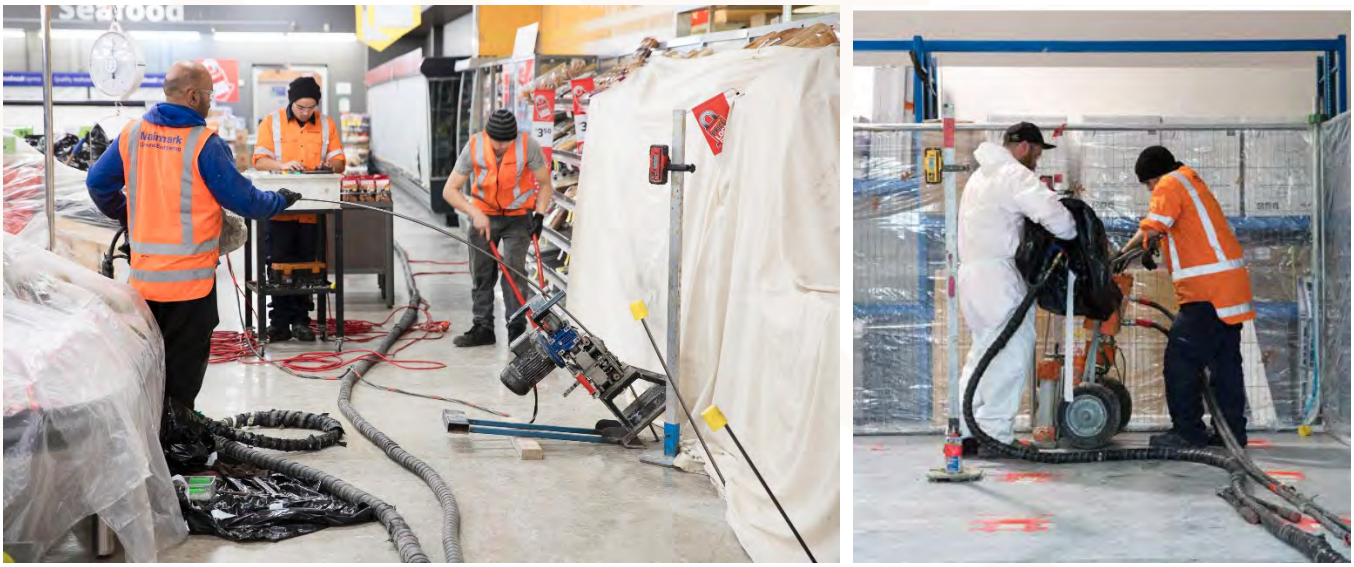


Figure 33 Installing injection tubes (left), injecting resin inside supermarket (right)

The buildings' load-bearing foundation areas were specified to a higher level of performance (i.e. requiring more reduction in settlement) than the floor areas, resulting in a design depth of treatment under the wall foundations of 7 metres, and 4 metres depth under the floor areas. During injection controlled ground heave was allowed in the areas of the building targeted for lifting as part of the relevelling requirements, and in other areas a 10mm heave cutoff critera was used.

Commercial Shopping Centre Test Panels

Prior to the main works a series of test panels (see Figure 34 below) were carried out to trial the effectiveness of the proposed mitigation method. Initially a test panel was installed inside one of the retail premises (under the existing shop floor), and another test panel was installed adjacent to the external (load bearing) wall of the same building. Three more trial panels were later installed adjacent to the external wall.

A project requirement was for the buildings to remain in continuous operation as 'big brand' retail outlets, with the works being carefully coordinated with the tenants. The initial internal (shop floor) test panel was installed over a series of night-time occupations. The internal production works were also carried out on the same basis, with minimised interruption to the continued occupation and operation of the buildings.

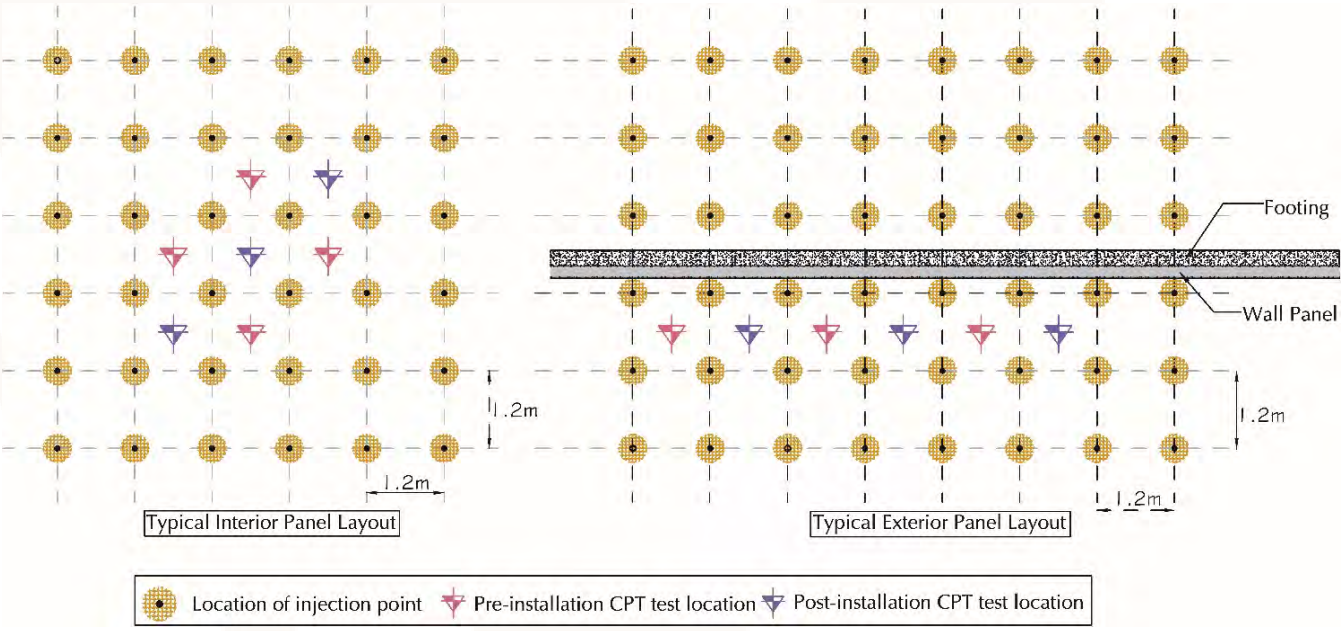


Figure 34 Typical test panel layouts commercial shopping centre (plan view)

The depth of injection, soil profiles, amount and type of material injected, as well as injection spacing and surface overburden varied between the test panels. Some of the variations and resulting improvements are detailed in Table 6.

Table 6 Summary of Trial Panel Configurations and Average Improvement Results

Trial	Treatment Zone	Injection Spacing	Material Volume Index*	Average Increase in Qc		Average Increase in q_{c1Ncs}
				(full depth)	(1- 4m)	(1- 4m)
Interior 1	1-4m	1.2m	1	75%	75%	50%
Exterior 1	1-7m	1.2m	1	55%	45%	45%
Exterior 2	1-7m	1.2m	1.7	60%	70%	50%
Exterior 3	1-7m	1.2m	2.4	10%	45%	40%
Exterior 4	1-7m	1.0m	2.5	60%	65%	40%

*volume of injected expanded material relative to Panel 1

Data from one of the exterior test panels is presented in Figure 35 below. For additional data plots refer to Traylen, van Ballegooy & Wentz (2016).

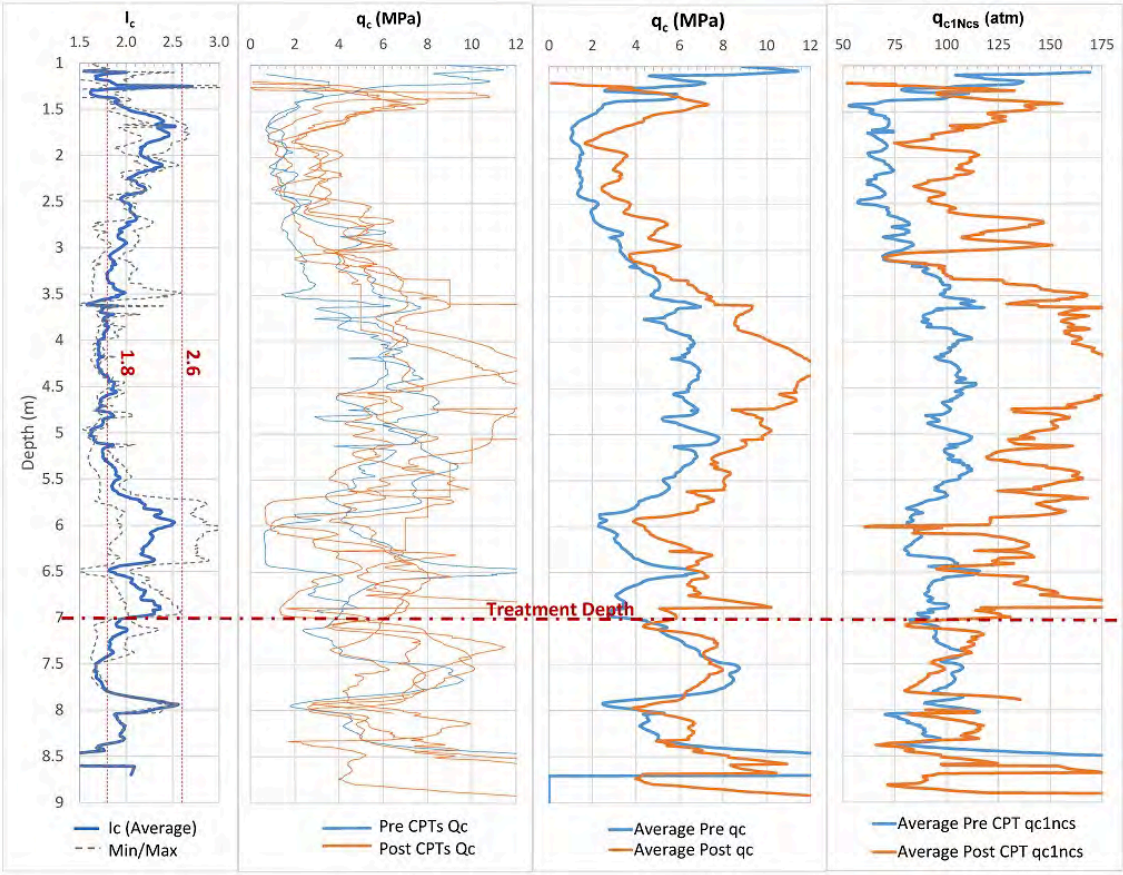


Figure 35 Commercial shopping centre exterior test panel 1

Commercial Shopping Centre Production Works

On the basis of the trial panels, a preferred methodology was selected, and production injection was carried out over the footprint of the building complex. (No additional surcharge loading other that the weight of the existing building was used for this project.) The client's geotechnical consultant had identified two distinct 'domains' within the project footprint, with the southern part of the site ('Domain 2') being siltier than the northern part. In Domain 1 a target performance specification based on q_{c1Ncs} was used, and in Domain 2 the requirement was to inject similar volumes of material as was achieved in Domain 1 (recognising that significant densification was less likely there, but also not required as the liquefaction performance of that part of the site was already significantly better).

The overall results were good, showing meaningful increases in average q_{c1Ncs} values and corresponding deceases in calculated settlements.

Table 7 Averaged Results Summary Commercial Shopping Centre

Dataset	% increase q_{c1Ncs} (atm)	% decrease in settlement (treatment zone)		
		SLS 0.19g/M6	ILS 0.3g/M6	ULS 0.35g/M7.5
All	37%	95%	85%	65%
Domain 1	37%	95%	85%	65%
Domain 2	23%	60%	45%	30%
Floor Areas	22%	70%	55%	30%
Wall Areas	41%	95%	85%	70%

The data in Table 7 above and Figure 36 below are from 30 CPT locations across the 5400m^2 footprint of the building complex.

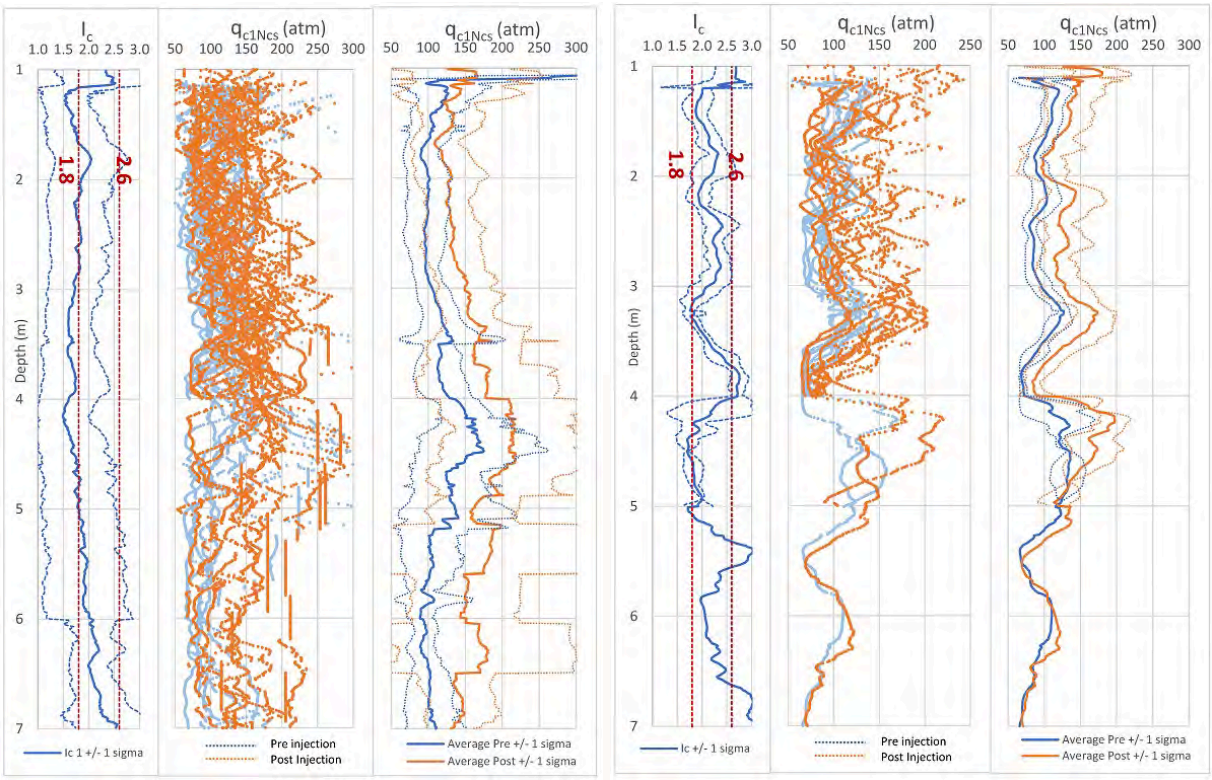


Figure 36 Pre- and post-injection data (production injections) Domain 1(left) and Domain 2 (right)

Commercial Shopping Centre Conclusion

The conclusion drawn from this ground improvement project was that the methodology is a viable technology for ground improvement, and is particularly useful for liquefaction mitigation beneath existing structures. Furthermore the low level of intrusion required to carry out the process was a major advantage for this operation, as the three retail outlets (including a busy supermarket) were able to continue trading, virtually uninterrupted, through the busy Christmas trading period.

Attachment 3

Peer Review Report

Rollins and Associates
Geotechnical Consultants

17 May 2017

John Scott
Geotechnical Advisor
Building Systems Performance Branch, Building, Resources and Markets
Ministry of Business, Innovation & Employment
Level 5, 15 Stout Street, P.O. Box 1473
Wellington 6143
New Zealand

Dear John,

The report is an excellent summary of a ground improvement trial project utilising resin injection. The report authors worked collaboratively with the peer reviewers and throughout the project addressed our comments and incorporated our suggestions. We consider that the results have been presented well and have been interpreted appropriately. The results of trials at all three sites demonstrate that the Mainmark resin injection system has been effective in improving the ground and thereby increasing the cyclic resistance of the soils. While the test results show a significant improvement, it is noted that the improvement mainly occurred in the sandy soils (with soil behaviour type index, I_C, values less than 2). In addition, the level of improvement was better at greater levels of confining stress typical beneath heavier commercial building structures. Therefore, judgement and care need to be exercised when extrapolating the results from these trials to other sites where the soils may be more silty or where the levels of confining stress are lower (such as residential building structures).

Sincerely,

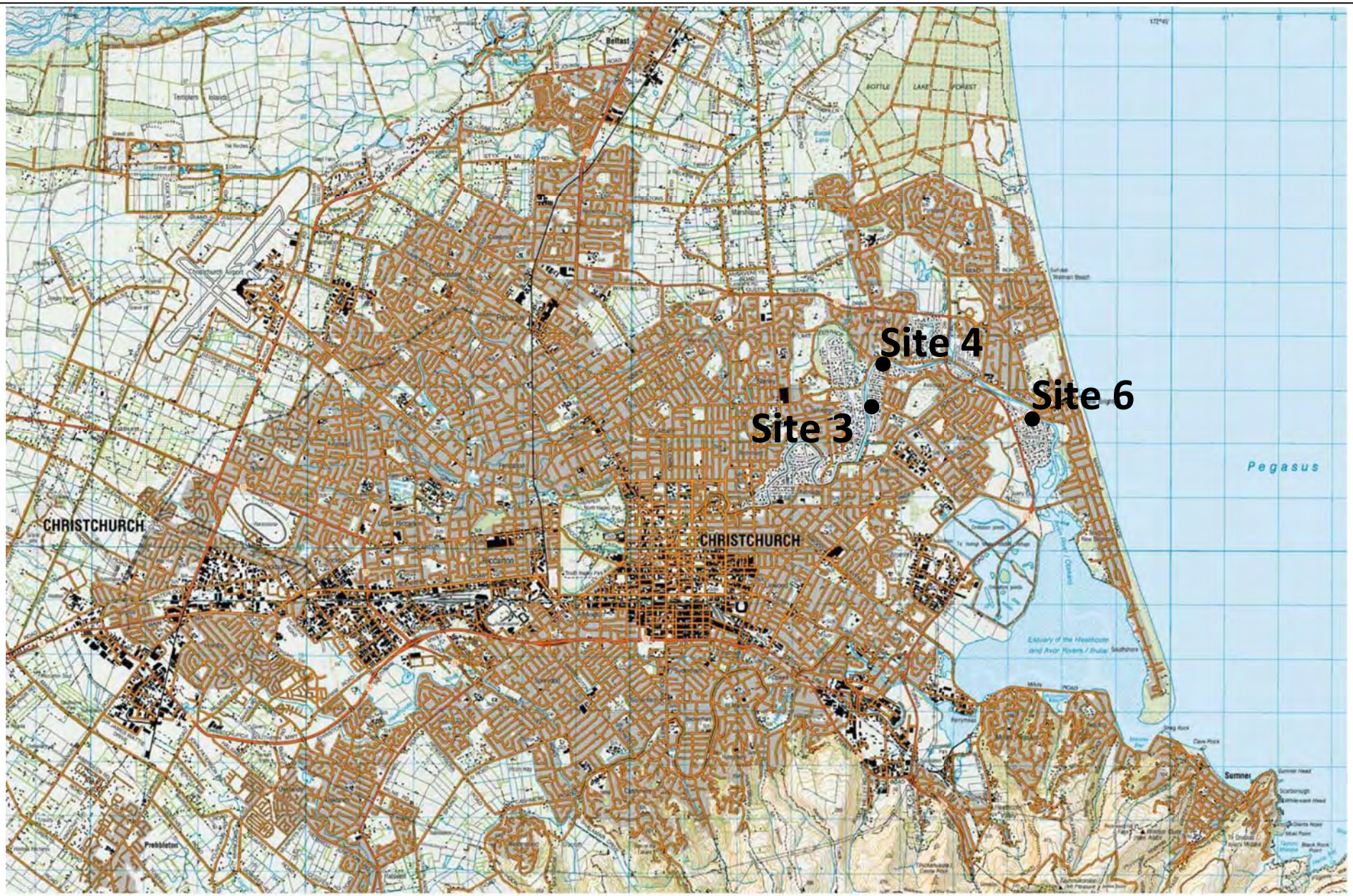
Kyle M. Rollins

Kyle Rollins, Ph.D.

Sjoerd Van Ballegooy, Ph.D.

1428 Timpanogos Dr., Provo, UT 84604

Appendix A
Site Locations



GEOTECH
consulting ltd

PROJECT:	DRAWING:
Mainmark Ground Engineering NZ Ltd Ground Improvement Trials	Site Locations

ISSUE	DATE	AMMENDMENT DETAILS	CHKD

SCALES:	DESIGNED	NJT	12/16
1:100,000 @ A4	DRAWN		
	CHECKED		

PROJECT No.	SHEET No.	ISSUE
4286	SLP1	A

Site Location
1:20,000

GEOTECH
consulting ltd

PROJECT:
Mainmark Ground Engineering NZ Ltd
Ground Improvement Trials

DRAWING:
Site Locations (Satellite)

ISSUE	DATE	AMMENDMENT DETAILS	CHKD

SCALES: 1:20,000 @ A4
DESIGNED: NJT 12/16
DRAWN:
CHECKED:

PROJECT No.	SHEET No.	ISSUE
4286	SLP2	A

Site Location
1:2000

Site Location
1:1000

GEOTECH
consulting ltd

PROJECT:	DRAWING:
Mainmark Ground Engineering NZ Ltd Ground Improvement Trials	Site 3 Breezes Road

ISSUE	DATE	AMMENDMENT DETAILS	CHKD

SCALES:	DESIGNED	NJT	12/16
As shown @ A4	DRAWN		
	CHECKED		

PROJECT No.	SHEET No.	ISSUE
4286	SLP3	A

Site Location
1:2000

Site Location
1:1000

GEOTECH
consulting ltd

PROJECT:	DRAWING:
Mainmark Ground Engineering NZ Ltd Ground Improvement Trials	Site 4 Ardrossan Street

ISSUE	DATE	AMMENDMENT DETAILS	CHKD

SCALES:	DESIGNED	NJT	12/16
As shown @ A4	DRAWN		
	CHECKED		

PROJECT No.	SHEET No.	ISSUE
4286	SLP4	A

Site Location
1:2000

Site Location
1:1000

GEOTECH
consulting ltd

PROJECT:	DRAWING:
Mainmark Ground Engineering NZ Ltd Ground Improvement Trials	Site 4 Ardrossan Street

ISSUE	DATE	AMMENDMENT DETAILS	CHKD

SCALES:	DESIGNED	NJT	12/16
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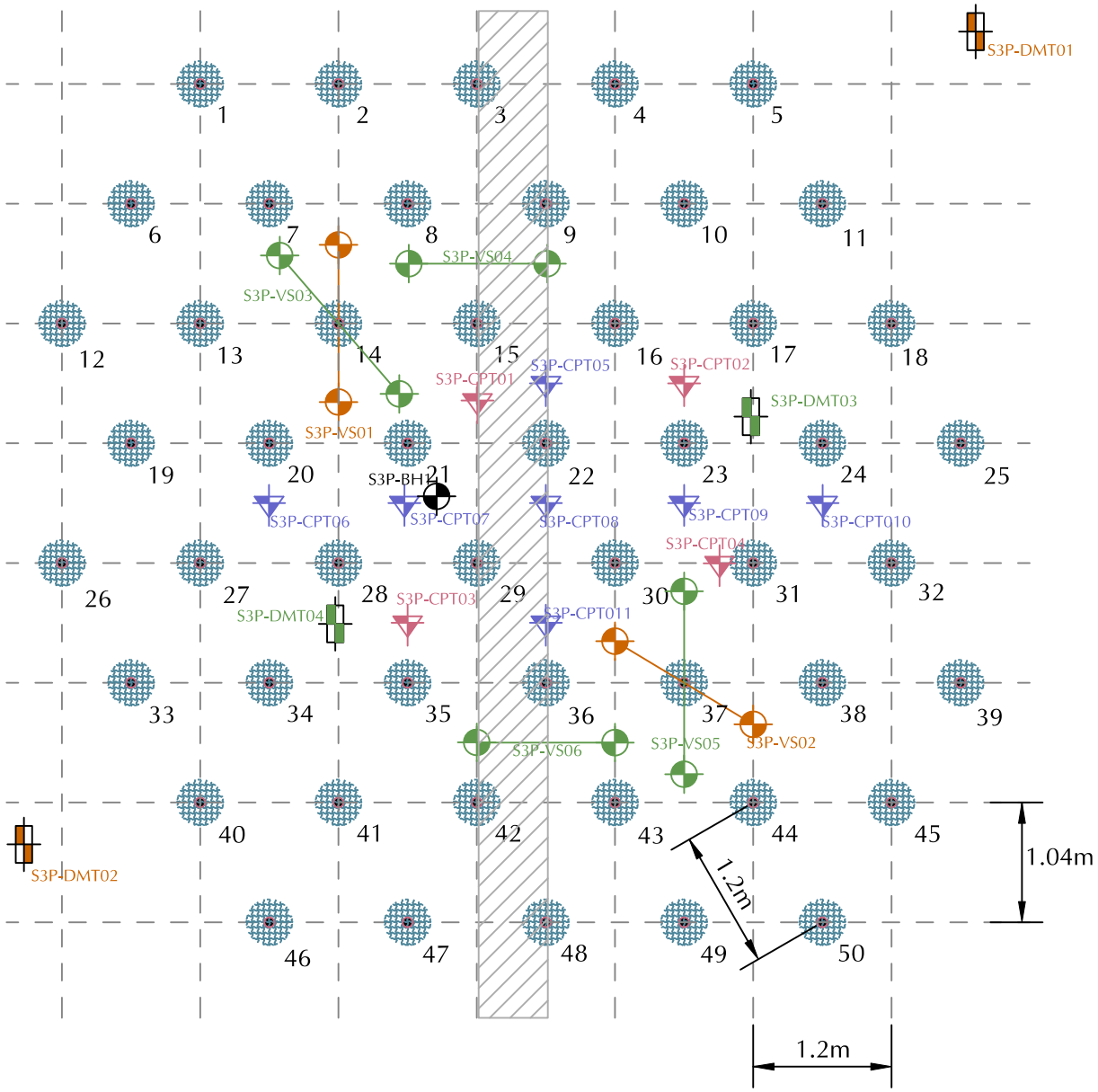
4286	SLP5	A

Appendix B
Test Panel Layouts



Legend

- Location of resin injection point
- Pre-installation CPT test location
- Post-installation CPT test location
- Pre-installation Vs pair
- Post-installation Vs pair
- Pre-installation DMT
- Post-installation DMT
- Borehole
- 28 kPa Strip Surcharge

GEOTECH consulting ltd

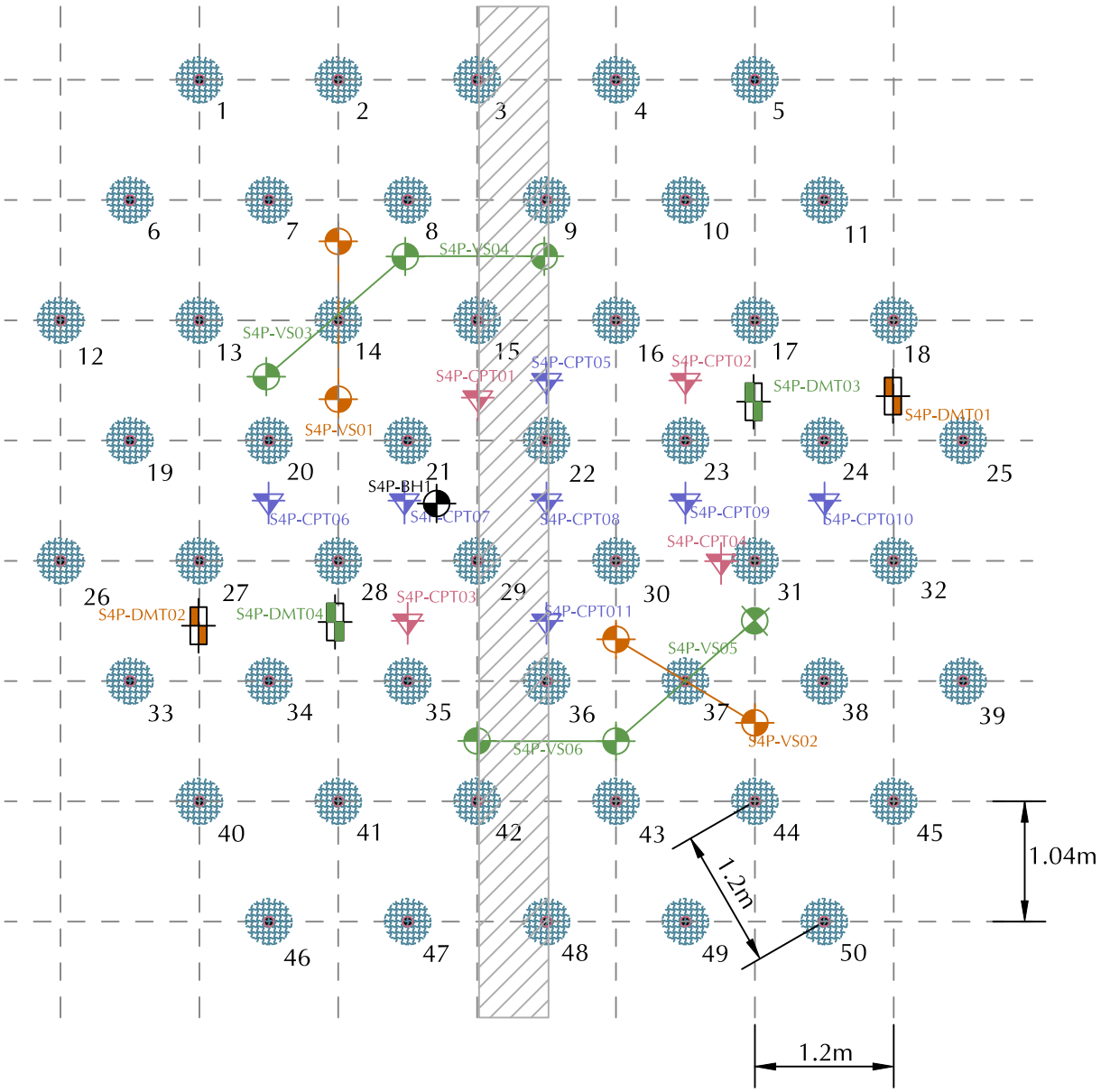
PROJECT:
Mainmark Ground Engineering NZ Ltd
Ground Improvement Trials

DRAWING:
Panel Layout Site 3 Breezes Road

ISSUE	DATE	AMMENDMENT DETAILS	CHKD

SCALES:			
n.t.s.	DESIGNED	NJT	01/17
	DRAWN		
	CHECKED		

PROJECT No.	SHEET No.	ISSUE
4286	PL3	A



Legend

- Location of resin injection point
- Pre-installation CPT test location
- Post-installation CPT test location
- Pre-installation Vs pair
- Post-installation Vs pair
- Pre-installation DMT
- Post-installation DMT
- Borehole
- 28 kPa Strip Surcharge

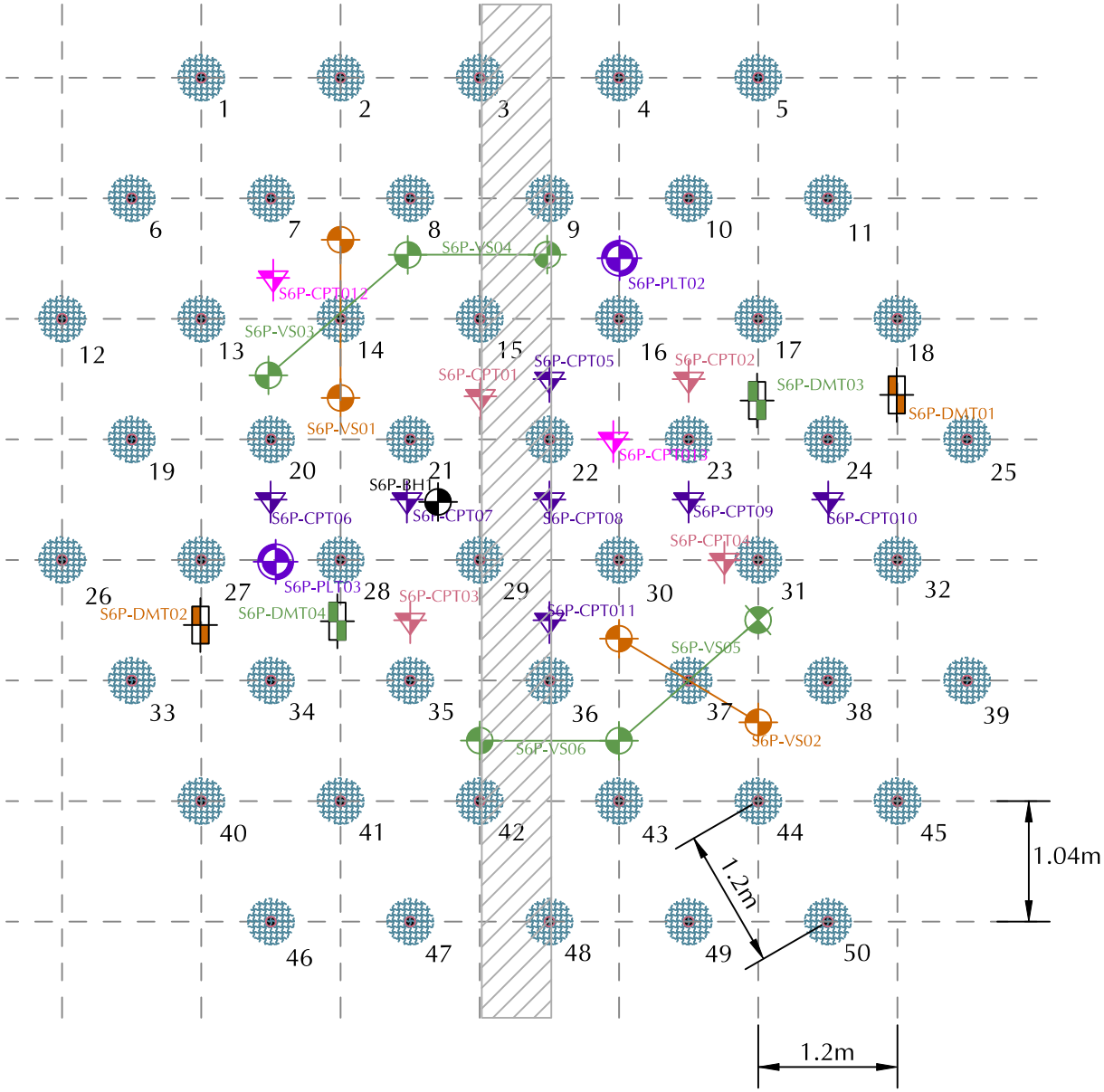
GEOTECH consulting ltd

PROJECT:	DRAWING:
Mainmark Ground Engineering NZ Ltd Ground Improvement Trials	Panel Layout Site 4 Ardrossan St

ISSUE	DATE	AMMENDMENT DETAILS	CHKD

SCALES:			
n.t.s.	DESIGNED	NJT	01/17
	DRAWN		
	CHECKED		

PROJECT No.	SHEET No.	ISSUE
4286	PL4	A



Legend

- Location of resin injection point
- Pre-installation CPT test location
- Post-installation CPT test location
- 90 Day CPT test location
- Pre-installation direct-push cross-hole (Vs Vp)
- Post-installation direct-push cross-hole (Vs Vp)
- Pre-installation DMT
- Post-installation DMT
- Plate Load Test
- Pre-installation Borehole
- 28 kPa Strip Surcharge

GEOTECH consulting ltd

PROJECT:	DRAWING:
Mainmark NZ Ltd Ground Improvement Trials	Panel Layout Site 6

ISSUE	DATE	AMMENDMENT DETAILS	CHKD

SCALES:			
NTS	DESIGNED	NJT	05/15
	DRAWN		
	CHECKED		

PROJECT No.	SHEET No.	ISSUE
4286	SK1	A

Appendix C
Borelogs, Laboratory Test Data

DRILLHOLE BORELOG

GEOTECH

Hole ID:	**S3P-BH01**
Sheet:	1 of 1
Date:	2/08/2016

Project No.: 4286	**Equipment:**	AMS VTR9100 - track	**G.L R.L:**	0.00m	**Logged By:**	NJT
Project: Red Zone Resin Trials	**Drilling Co:**	McMillan Drilling	**Max Depth:**	8.10m	**Checked By:**	NJT
Client: Mainmark Ground Engineering (NZ) Ltd	**Operator:**	R. Conkie	**Inclination:**	90°	**Sampled By:**	NJT

North (m): - **East (m):** - **Grid:** - **Location:** Refer to the site plan.

Geological Formation	STRATA DESCRIPTION	Depth	Classification Symbol	TCR (%)	Samples	SPT (blows/mm)
	GRAVEL with minor Silt; tan. Coarse Gravel, medium dense, wet. (FILL)		OL	100%		
	Clayey SILT; dark brown. Damp. (Topsoil)		ML			
	Sandy SILT; brown. Damp		SM			
	Silty SAND; brown. Medium Sand, damp.	1.0	ML			
	Sandy SILT; brown. Damp					
	Silty SAND to SILT with some Sand and minor Clay; brown. Damp.		SM	100%	1.30m C	
	Silty SAND; brownish grey. Fine to medium Sand, damp.	2.0			1.75m C	
	SAND; brown. Medium to coarse Sand, wet.	3.0	SP	100%		
					3.50m C	
		4.0		100%		
	SAND; grey. Fine to medium Sand, wet.					
		5.0		100%	5.00m C	
	SAND; grey. Medium to coarse Sand, wet.					
		6.0		60%		
		7.0			6.70m C	
				100%		
		8.0				

EOH: 8.10m

Remarks: No static water level recorded
Samples in core boxes
400 liters water added

DRILLHOLE BORELOG

GEOTECH

Hole ID:	**S4P-BH01**
Sheet:	1 of 1
Date:	2/08/2016

Project No.: 4286	**Equipment:**	AMS VTR9100 - track	**G.L R.L:**	0.00m	**Logged By:**	NJT
Project: Red Zone Resin Trials	**Drilling Co:**	McMillan Drilling	**Max Depth:**	8.00m	**Checked By:**	NJT
Client: Mainmark Ground Engineering (NZ) Ltd	**Operator:**	R. Conkie	**Inclination:**	90°	**Sampled By:**	NJT

North (m): - **East (m):** - **Grid:** - **Location:** Refer to the site plan

Geological Formation	STRATA DESCRIPTION	Graphic Log	Depth	Classification Symbol	Piezometer & Water Levels	TCR (%)	Drill Method	Samples	Tests	SPT (blows/mm)
	Silty SAND; brown. Fine to medium Sand, damp to wet.		1.0	SM		100%				
	SILT; grey. Damp.			ML						
	Silty SAND to Sandy SILT; grey. Fine to medium Sand, damp.		2.0	SM		100%		2.00m C		
	Silty SAND; greyish brown. Fine to medium Sand, damp.									
	Silty SAND; brownish grey. Fine to medium Sand, damp.		3.0	SM		90%				
	SAND; grey. Fine to medium Sand, damp.		4.0	SP						
	Silty SAND; grey. Fine to medium Sand, damp.			SM		70%		4.20m C		
	No Sample.	C/L								
	SAND; grey. Medium to coarse Sand, damp.		5.0	SP						
	SAND; grey. Fine to medium Sand, damp.			SP		75%		5.00m C		
	SAND; grey. Medium to coarse Sand, damp.		6.0			60%		6.50m C		
	Silty SAND; grey. Fine to coarse Sand, damp.		7.0	SM		100%				
	SAND with some Gravel; grey. Medium to coarse Sand, damp.		8.0	SP						

EOH: 8.00m

Remarks: No static water level recorded
Samples in core boxes
400 liters water added

DRILLHOLE BORELOG

GEOTECH

Hole ID:	**S6P-BH01**
Sheet:	1 of 1
Date:	3/08/2016

Project No.: 4286	**Equipment:**	AMS VTR9100 - track	**G.L R.L:**	0.00m	**Logged By:**	NJT
Project: Red Zone Resin Trials	**Drilling Co:**	McMillan Drilling	**Max Depth:**	8.00m	**Checked By:**	NJT
Client: Mainmark Ground Engineering (NZ) Ltd	**Operator:**	R. Conkie	**Inclination:**	90°	**Sampled By:**	NJT

North (m): - **East (m):** - **Grid:** - **Location:** Refer to the site plan

Geological Formation	STRATA DESCRIPTION	Graphic Log	Depth	Classification Symbol	Piezometer & Water Levels	TCR (%)	Drill Method	Samples	Tests	SPT (blows/mm)
	Silty GRAVEL. Coarse Gravel (FILL)									
	Clayey SILT (Topsoil) and plastic (FILL)									
	SAND; brown. Fine to coarse Sand, damp.					80%				
	Silty SAND to Sandy SILT; brown. Fine to medium Sand, damp.		1.0	SP						
	Silty SAND; brown. Fine to medium Sand.			SM						
	SAND; brownish grey. Fine to coarse Sand, damp.		2.0			95%		1.50m B		
	SAND; grey. Medium to coarse Sand, damp.		3.0			95%				
			4.0			80%		4.00m B		
				SW						
			5.0			60%		4.80m B		
			6.0			95%				
			7.0					7.00m B		
						60%				
			8.0							

EOH: 8.00m

Remarks: No static water level recorded
Samples in core boxes
400 liters water added

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: 16/1801

Date: 12 August 2016

TEST REPORT – BREEZES ROAD INVESTIGATIONS

Client Details:	Geotech Consulting Ltd, P.O. Box 130-122, Christchurch	Attention:	N. Traylen
Job Description:	Breezes Road Investigations		
Sample Description:	As Below	Client Job No:	Not Stated
Sample Source:	S3P-BH01	Reference No:	Not Stated
Date & Time Sampled:	2-Aug-16	Sampled By:	Unknown
Sample Method:	Borehole	Date Received:	9-Aug-16

Sample Source	Sample Description	% Passing 75µm Sieve	% Passing 63µm Sieve	Water Content As Received (%)	Plasticity Index Fraction Tested	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
S3P-BH01 @ 1.3m (Sample 1)	SILT with some sand and minor clay	91	87	27.4	Whole Soil	31	23	8
S3P-BH01 @ 1.75m (Sample 2)	Silty SAND	39	29	30.6	-	-	-	-
S3P-BH01 @ 3.5m (Sample 3)	SAND with minor gravel & trace of silt	2	2	20.5	-	-	-	-
S3P-BH01 @ 6.7m (Sample 5)	SAND with trace of gravel & trace of silt	1	1	22.9	-	-	-	-

Note: The samples were received in a natural state.

Test Methods:

- Particle Size Analysis - NZS 4402:1986, Test 2.8.1
- Plasticity Index - NZS 4402:1986, Test 2.2, 2.3 & 2.4
- Water Content - NZS 4402:1986, Test 2.1

Note:

- *Information contained in this report which is Not IANZ Accredited relates to the sample descriptions based on NZ Geotechnical Society Guidelines 2005 and sampling.*
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Tested By: A.P. Julius **Date:** 10 to 12-Aug-16

Checked By:

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Reference No: 16/1801

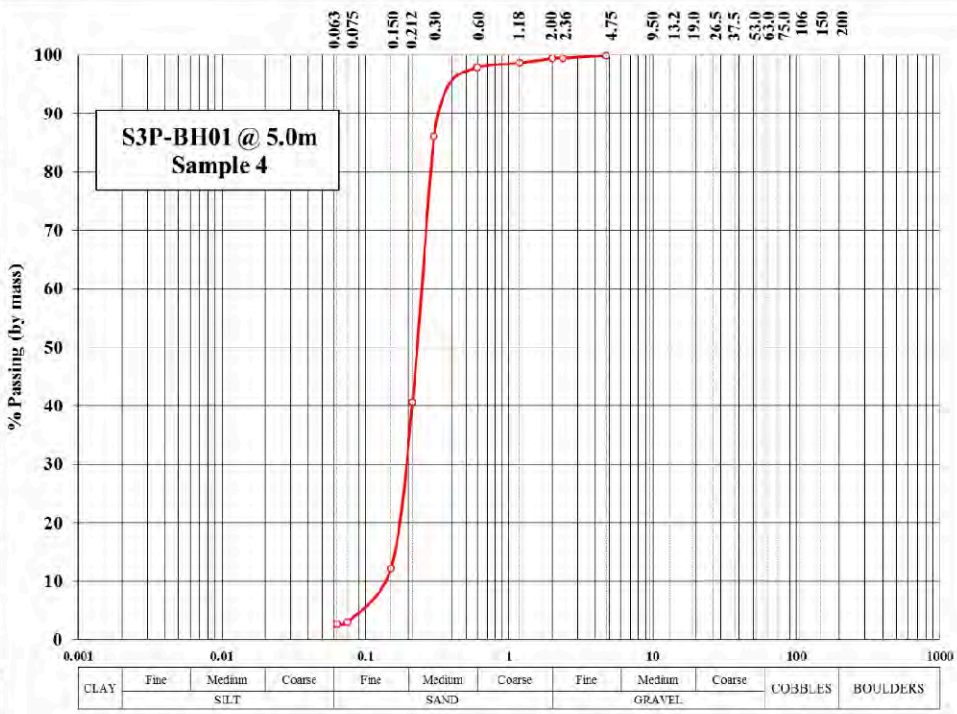
Date: 12 August 2016

TEST REPORT – BREEZES ROAD INVESTIGATIONS

Client Details:	**Geotech Consulting Ltd, P.O. Box 130-122, Christchurch**	**Attention:**	**N. Traylen**
Job Description:	**Breezes Road Investigations**		
Sample Description:	**SAND with trace of gravel and trace of silt**	**Client Job No:**	**Not Stated**
Sample Source:	**S3P-BH01 @ 5.0m (Sample #4)**	**Reference No:**	**Not Stated**
Date & Time Sampled:	**2-Aug-16**	**Sampled By:**	**Unknown**
Sample Method:	**Borehole**	**Date Received:**	**9-Aug-16**

PARTICLE SIZE ANALYSIS (NZS 4402:1986, Test 2.8.1)

Test Sieve (mm)	% Passing (by mass)
37.5	
26.5	
19.0	
13.2	
9.50	
4.75	100
2.36	99
2.00	99
1.18	99
0.60	98
0.30	86
0.212	41
0.150	12
0.075	3
0.063	3



The sample was received in a natural state. The percentage passing the 63μm test sieve was obtained by difference.

WATER CONTENT RESULTS - NZS 4402:1986, Test 2.1	
Water Content: (As Received)	**18.1 %**
Note: The sample was received in a natural state.	

Note:

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Tested By: A.P. Julius **Date:** 10 to 12-Aug-16

Checked By:

Approved Signatory

A.P. Julius
Laboratory Manager

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Reference No: 16/1802

Date: 12 August 2016

TEST REPORT – ARDROSSEN STREET INVESTIGATIONS

Client Details:	**Geotech Consulting Ltd, P.O. Box 130-122, Christchurch**	**Attention:**	**N. Traylen**
Job Description:	**Ardrossen Street Investigations**		
Sample Description:	**As Below**	**Client Job No:**	**Not Stated**
Sample Source:	**S4P-BH01**	**Reference No:**	**Not Stated**
Date & Time Sampled:	**2 & 5-Aug-16**	**Sampled By:**	**Unknown**
Sample Method:	**Borehole**	**Date Received:**	**9-Aug-16**

Sample Source	Sample Description	% Passing 75µm Sieve	% Passing 63µm Sieve	Water Content As Received (%)	Plasticity Index Fraction Tested	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
S4P-BH01 @ 2.0m (Sample 1)	Sandy SILT with some clay	78	72	36.4	Whole Soil	46	27	19
S4P-BH01 @ 4.2m (Sample 2)	Silty SAND	45	34	26.7	-425 µm	Not Applicable	Non-Plastic	Non-Plastic
S4P-BH01 @ 6.5m (Sample 4)	SAND with trace of gravel & trace of silt	2	2	22.5	-	-	-	-

Note: The samples were received in a natural state.

Test Methods:

- Particle Size Analysis - NZS 4402:1986, Test 2.8.1
- Plasticity Index - NZS 4402:1986, Test 2.2, 2.3 & 2.4
- Water Content - NZS 4402:1986, Test 2.1

Note:

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Tested By: A.P. Julius **Date:** 10 to 12-Aug-16

Checked By:

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Reference No: 16/1802

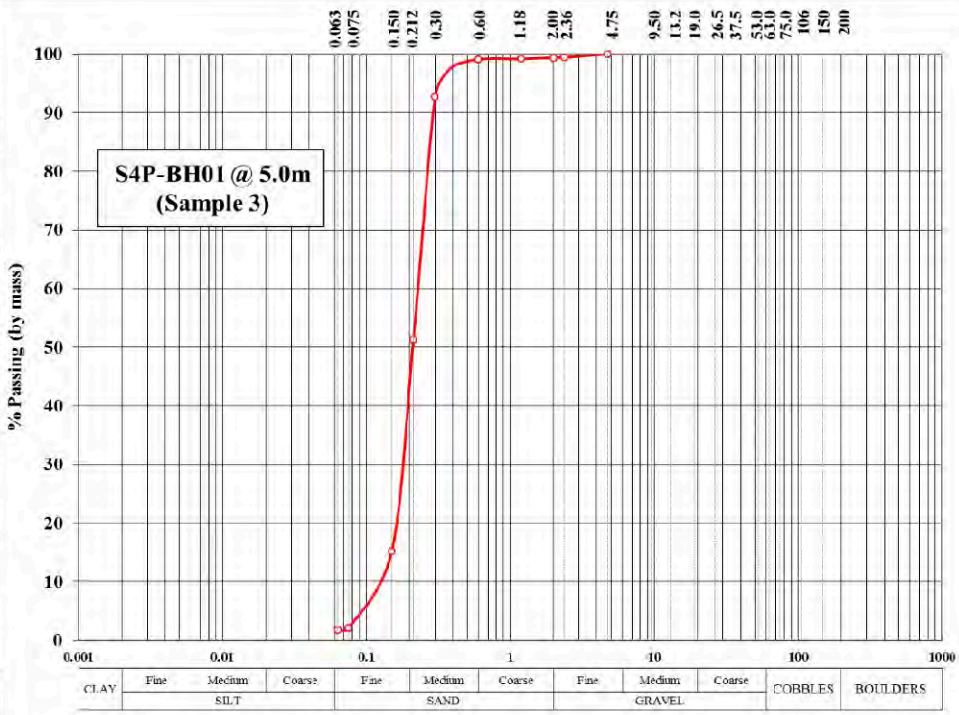
Date: 12 August 2016

TEST REPORT – ARDROSSEN STREET INVESTIGATIONS

Client Details:	**Geotech Consulting Ltd, P.O. Box 130-122, Christchurch**	**Attention:**	**N. Traylen**
Job Description:	**Ardrossen Street Investigations**		
Sample Description:	**SAND with trace of gravel & trace of silt**	**Client Job No:**	**Not Stated**
Sample Source:	**S4P-BH01 @ 5.0m (Sample 3)**	**Reference No:**	**Not Stated**
Date & Time Sampled:	**5-Aug-16**	**Sampled By:**	**Unknown**
Sample Method:	**Borehole**	**Date Received:**	**9-Aug-16**

PARTICLE SIZE ANALYSIS (NZS 4402:1986, Test 2.8.1)

Test Sieve (mm)	% Passing (by mass)
37.5	
26.5	
19.0	
13.2	
9.50	
4.75	100
2.36	99
2.00	99
1.18	99
0.60	99
0.30	93
0.212	51
0.150	15
0.075	2
0.063	2



The sample was received in a natural state. The percentage passing the 63μm test sieve was obtained by difference.

WATER CONTENT RESULTS - NZS 4402:1986, Test 2.1	
Water Content: (As Received)	**22.1 %**
Note: The sample was received in a natural state.	

Note:

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Tested By: A.P. Julius **Date:** 10 to 12-Aug-16

Checked By:

Approved Signatory

A.P. Julius
Laboratory Manager

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Reference No: 16/1803

Date: 12 August 2016

TEST REPORT – ORARI STREET INVESTIGATIONS

Client Details:	**Geotech Consulting Ltd, P.O. Box 130-122, Christchurch**	**Attention:**	**N. Traylen**
Job Description:	**Orari Street Investigations**		
Sample Description:	**As Below**	**Client Job No:**	**Not Stated**
Sample Source:	**S6P-BH01**	**Reference No:**	**Not Stated**
Date & Time Sampled:	**5-Aug-16**	**Sampled By:**	**Unknown**
Sample Method:	**Borehole**	**Date Received:**	**9-Aug-16**

Sample Source	Sample Description	% Passing 75µm Sieve	% Passing 63µm Sieve	Water Content As Received (%)
S6P-BH01 @ 1.5m (Sample 1)	SAND with some silt	21	14	27.7
S6P-BH01 @ 4.8 - 6.0m (Sample 3)	SAND	1	0	26.7
S6P-BH01 @ 7.0m (Sample 4)	SAND	0	0	23.4

Note: The samples were received in a natural state.

Test Methods:

- Particle Size Analysis - NZS 4402:1986, Test 2.8.1
- Water Content - NZS 4402:1986, Test 2.1

Note:

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Tested By: A.P. Julius **Date:** 10 to 12-Aug-16

Checked By:

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

Reference No: 16/1803

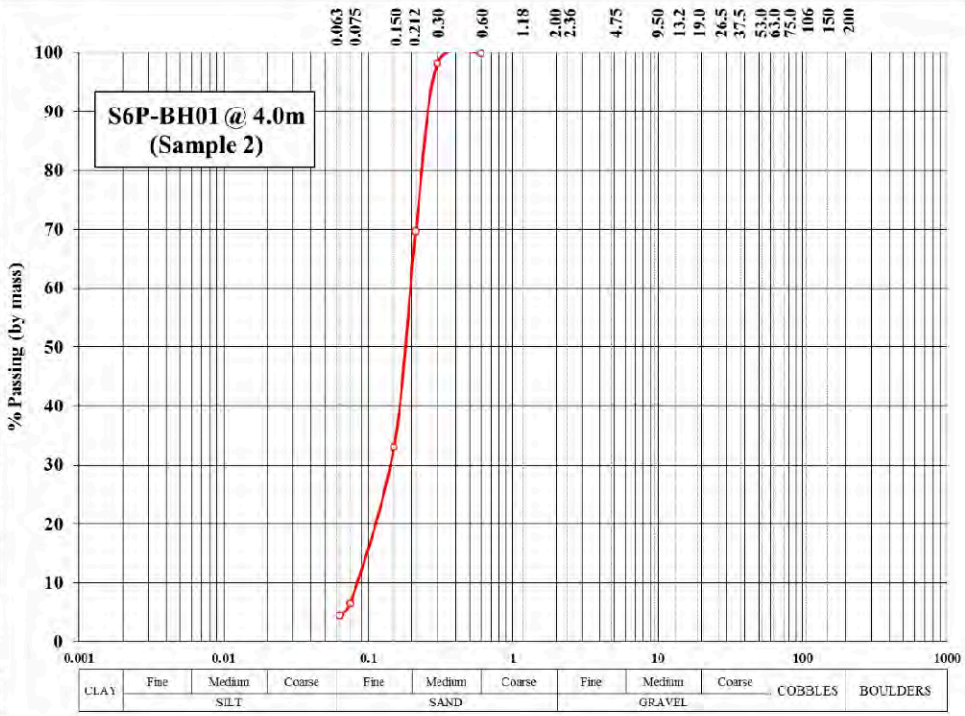
Date: 12 August 2016

TEST REPORT – ORARI STREET INVESTIGATIONS

Client Details:	**Geotech Consulting Ltd, P.O. Box 130-122, Christchurch**	**Attention:**	**N. Traylen**
Job Description:	**Orari Street Investigations**		
Sample Description:	**SAND with trace of silt**	**Client Job No:**	**Not Stated**
Sample Source:	**S6P-BH01 @ 4.0m (Sample 2)**	**Reference No:**	**Not Stated**
Date & Time Sampled:	**5-Aug-16**	**Sampled By:**	**Unknown**
Sample Method:	**Borehole**	**Date Received:**	**9-Aug-16**

PARTICLE SIZE ANALYSIS (NZS 4402:1986, Test 2.8.1)

Test Sieve (mm)	% Passing (by mass)
37.5	
26.5	
19.0	
13.2	
9.50	
4.75	
2.36	
2.00	
1.18	
0.60	100
0.30	98
0.212	70
0.150	33
0.075	7
0.063	4



The sample was received in a natural state. The percentage passing the 63µm test sieve was obtained by difference.

WATER CONTENT RESULTS - NZS 4402:1986, Test 2.1	
Water Content: (As Received)	**24.1 %**
Note: The sample was received in a natural state.	

Note:

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Tested By: A.P. Julius **Date:** 10 to 12-Aug-16

Checked By:

Approved Signatory

A.P. Julius
Laboratory Manager

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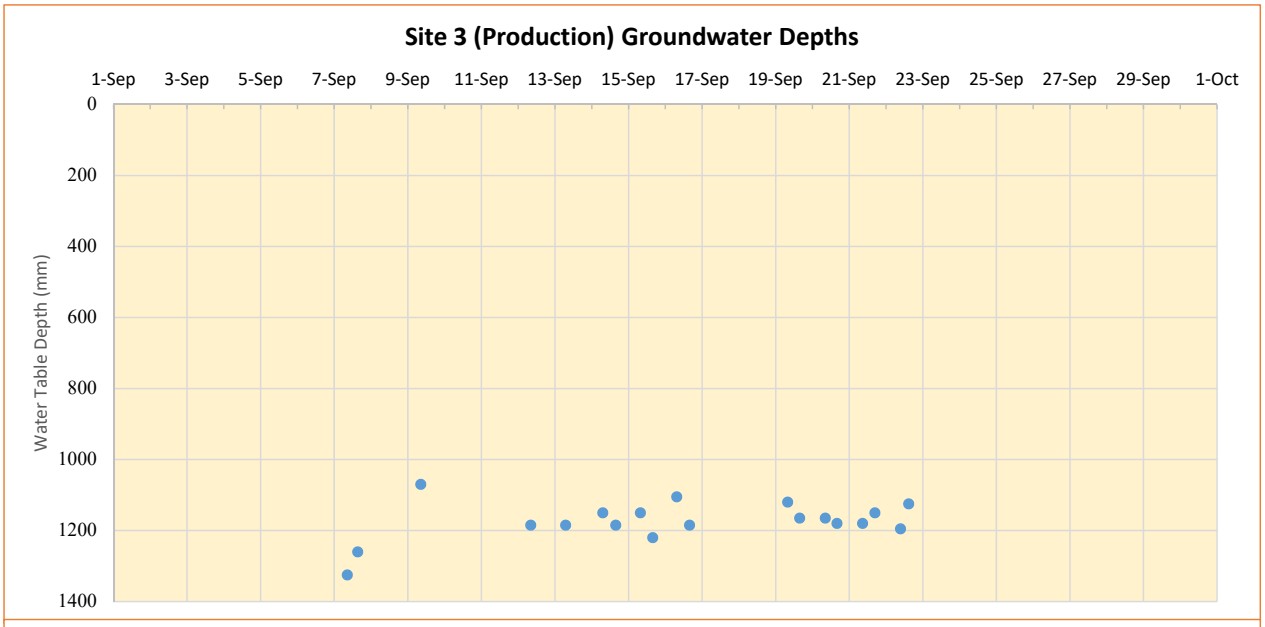
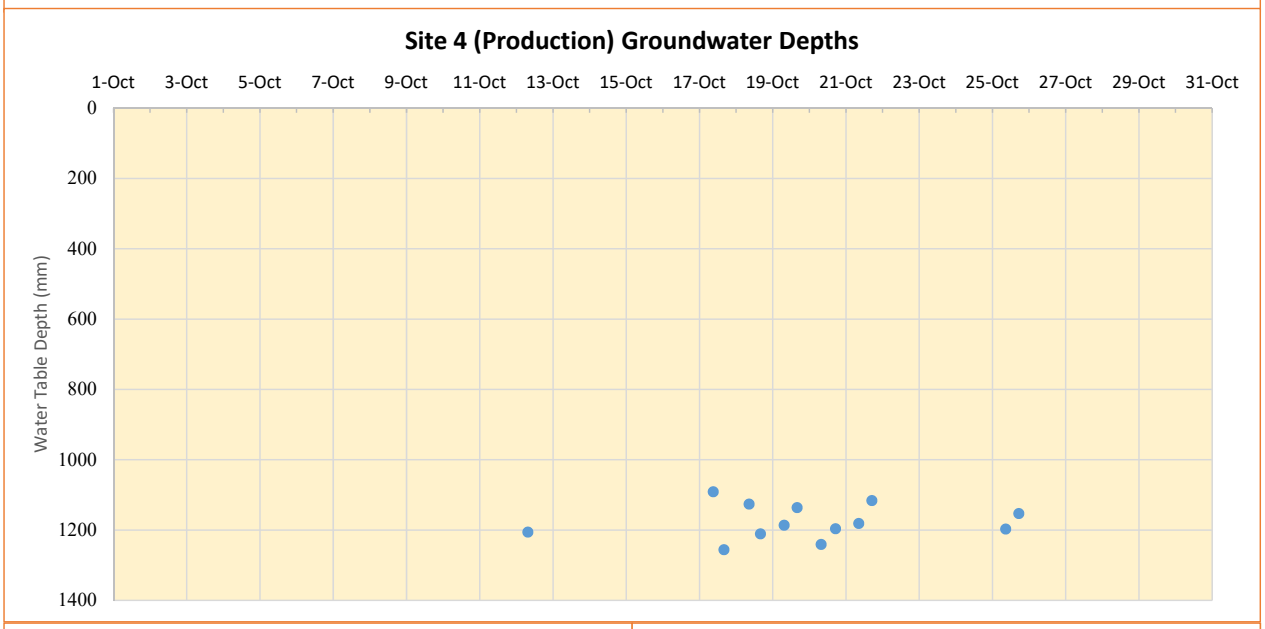
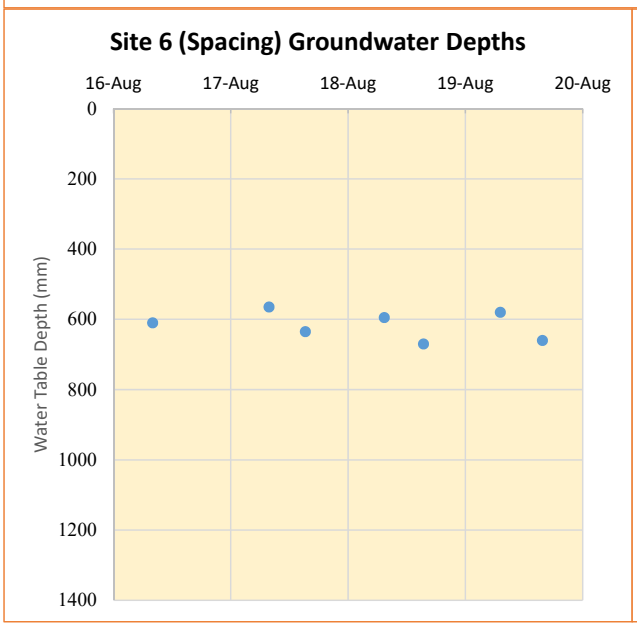
Accreditation No: 434

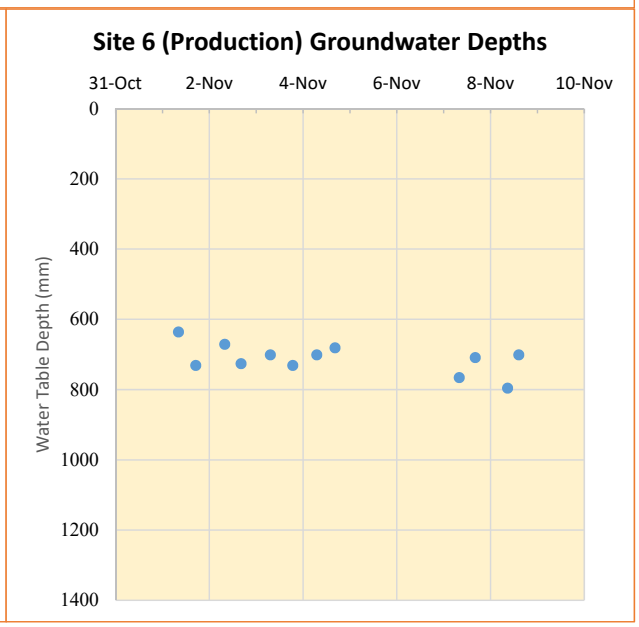
Appendix D
Groundwater

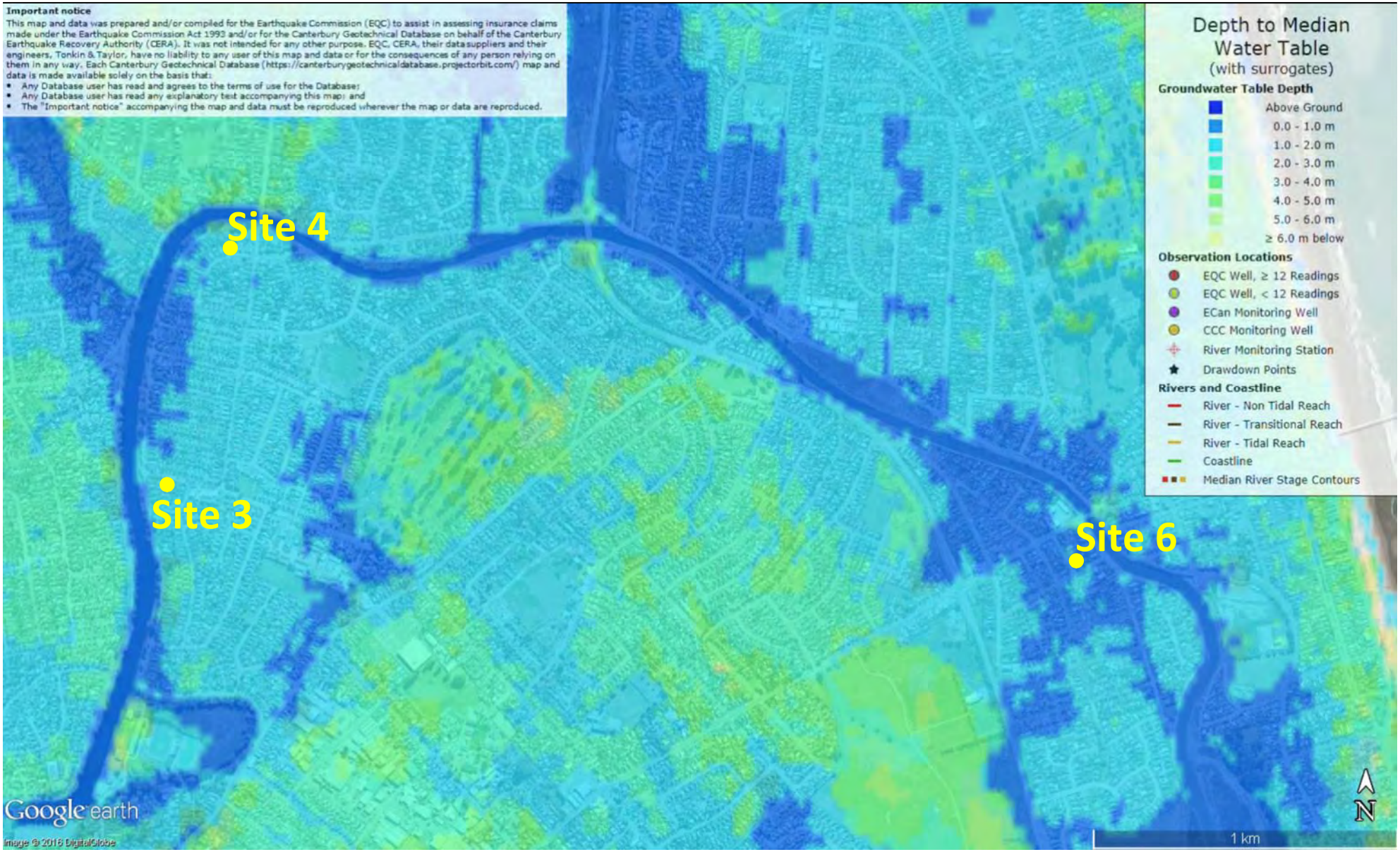
Groundwater Data Collected August to November 2016











GEOTECH consulting ltd

PROJECT:	DRAWING:
Mainmark Ground Engineering NZ Ltd Ground Improvement Trials	GNS Groundwater Model Median Depths to Groundwater

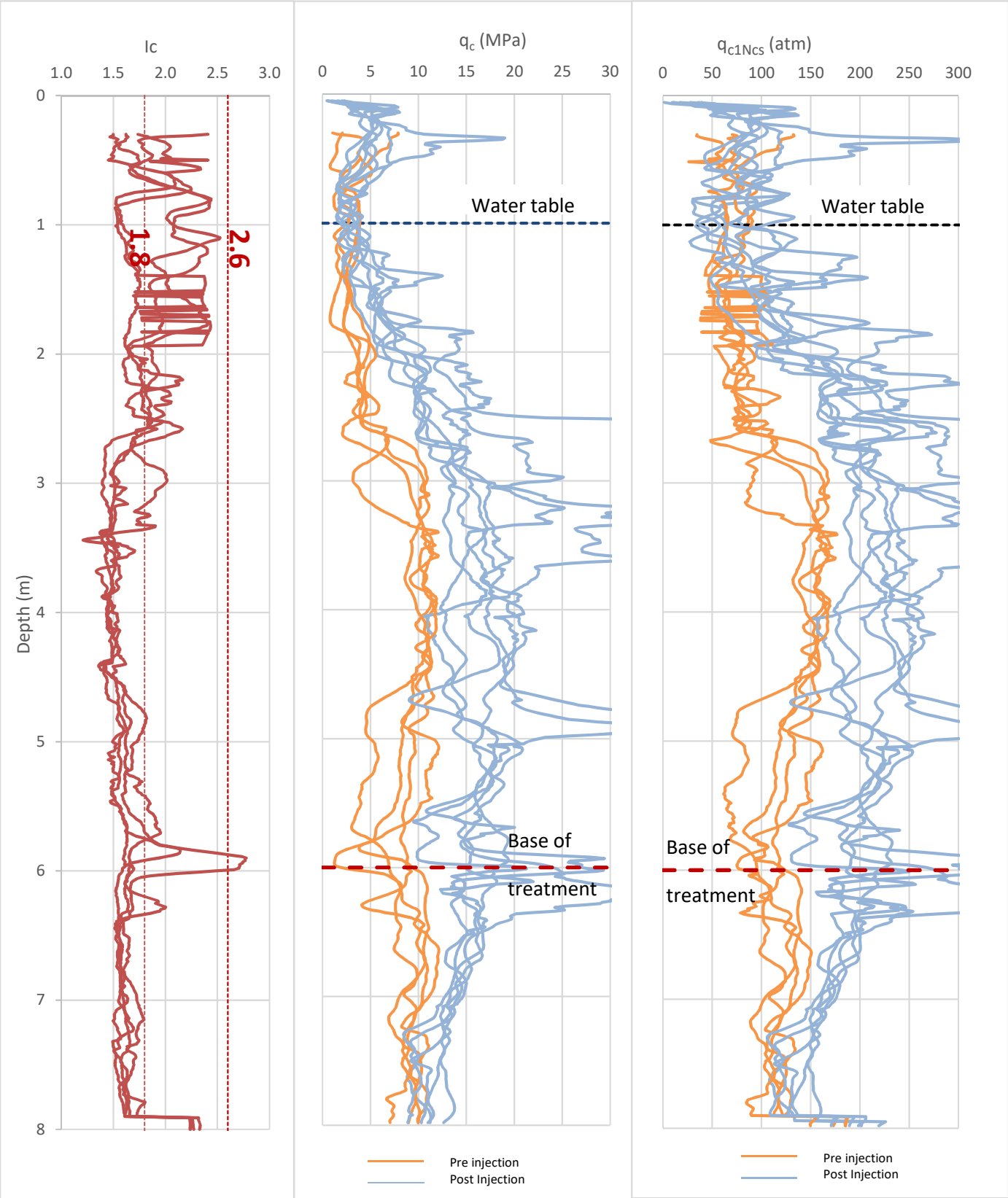
ISSUE	DATE	AMMENDMENT DETAILS	CHKD

SCALES:	DESIGNED	NJT	02/17
1:17,500 @ A4	DRAWN		
	CHECKED		

PROJECT No.	SHEET No.	ISSUE
4286	GW1	A

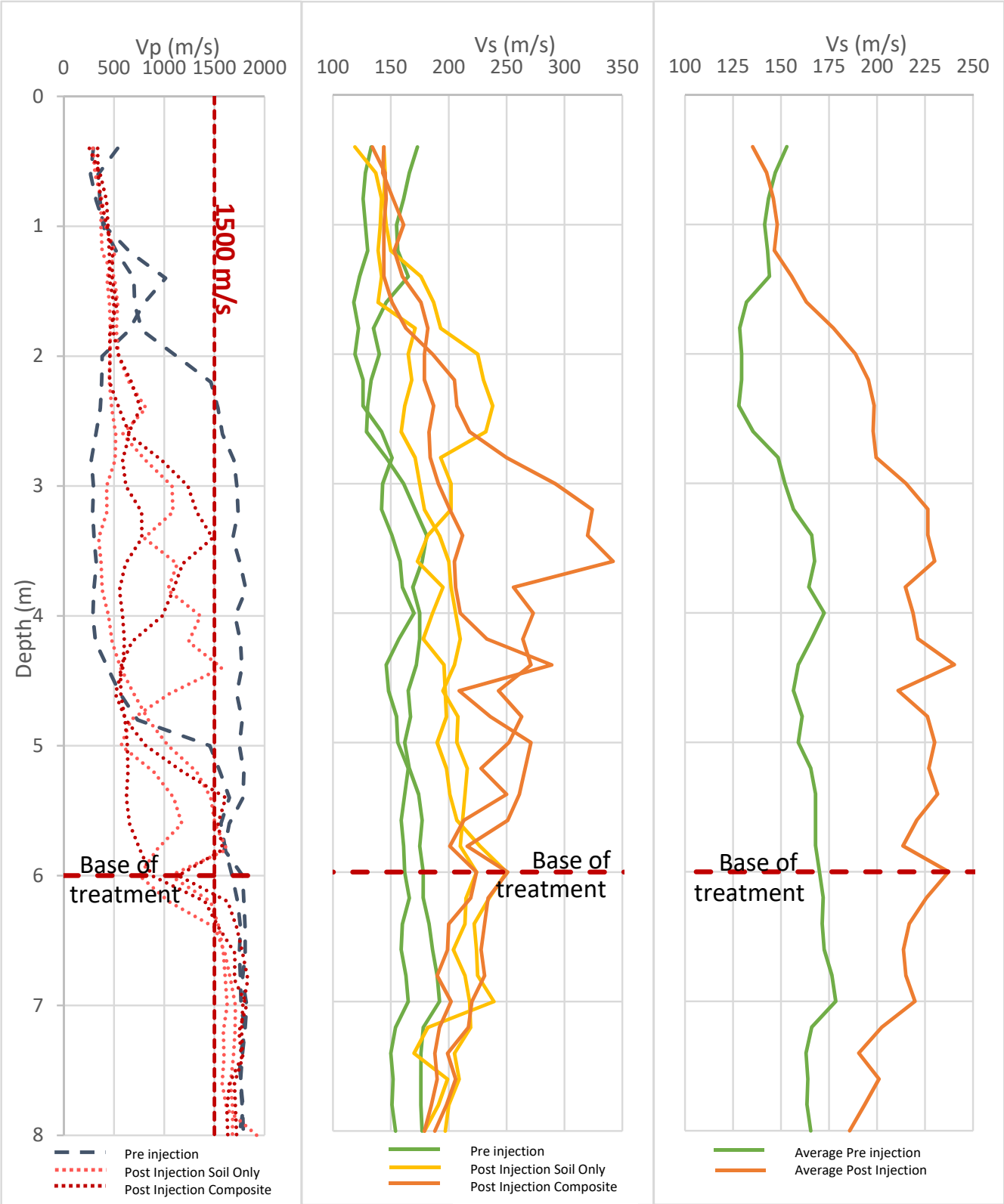
Appendix E
Test Results Site 3

GEOTECH	CONE PENETROMETER TEST DATA			
	GEOTECH CONSULTING LTD			
	Project:	Red Zone Trials Polyurethane Injection	Loc No:	**Production Panel 1**
	Client:	Mainmark Ground Engineering (NZ) Ltd	Job No:	4286 Site 3



Average pre qc:	7	MPa	Average post q_{c1Ncs}:	113	atm
Average post qc:	13	MPa	Average pre q_{c1Ncs}:	190	atm

GEOTECH	GEOPHYSICAL TESTING DATA			
	GEOTECH CONSULTING LTD			
	Project:	Red Zone trials Resin Injection	Loc No:	**Production Panel 1**
	Client:	Mainmark Ground Engineering Ltd	Job No:	4286 Site 3

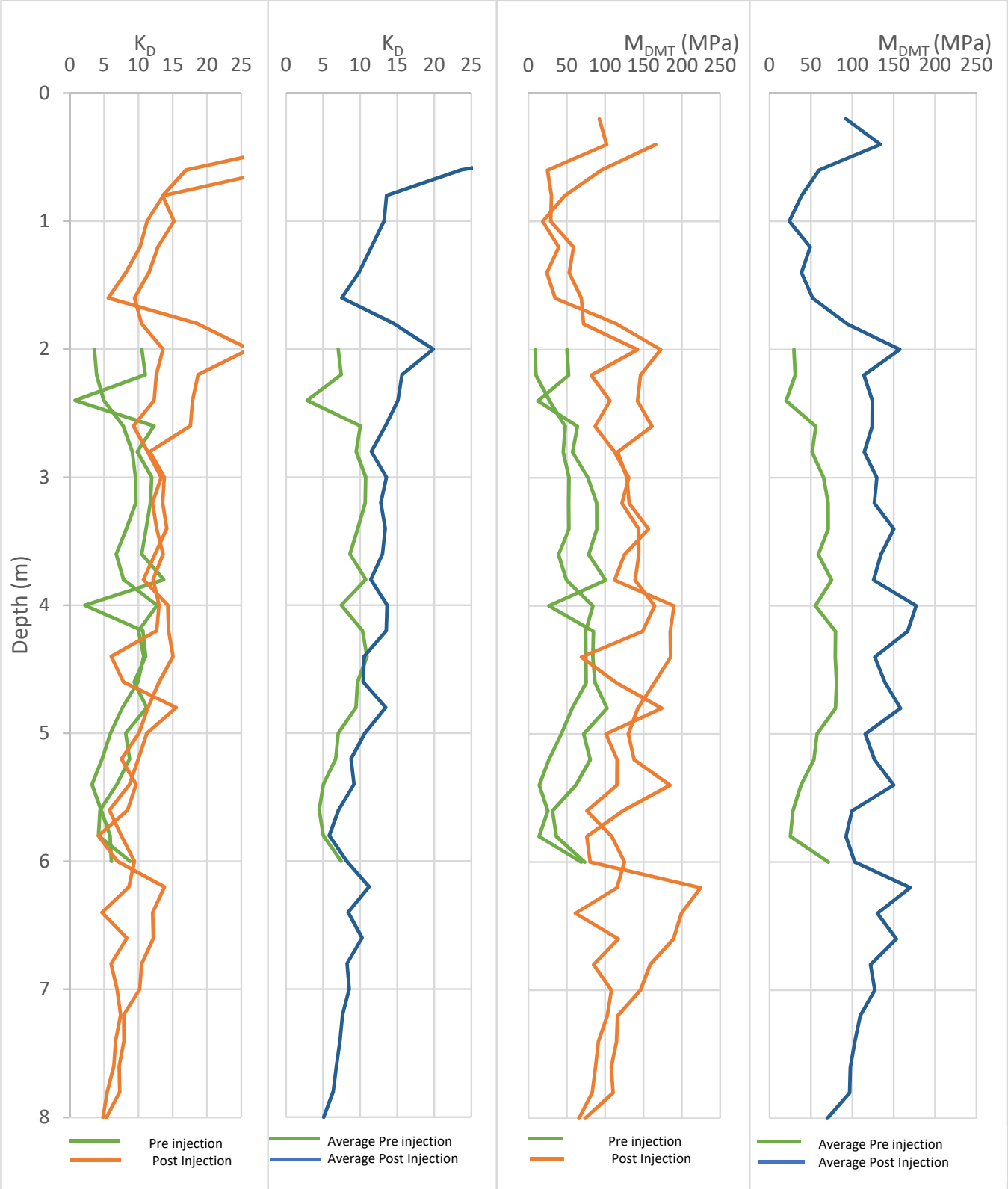


Average pre-inj:	153 m/s	Average post (composite):	221 m/s
Average post (soil):	192 m/s	Average post (overall):	206 m/s

DILATOMETER TESTING DATA

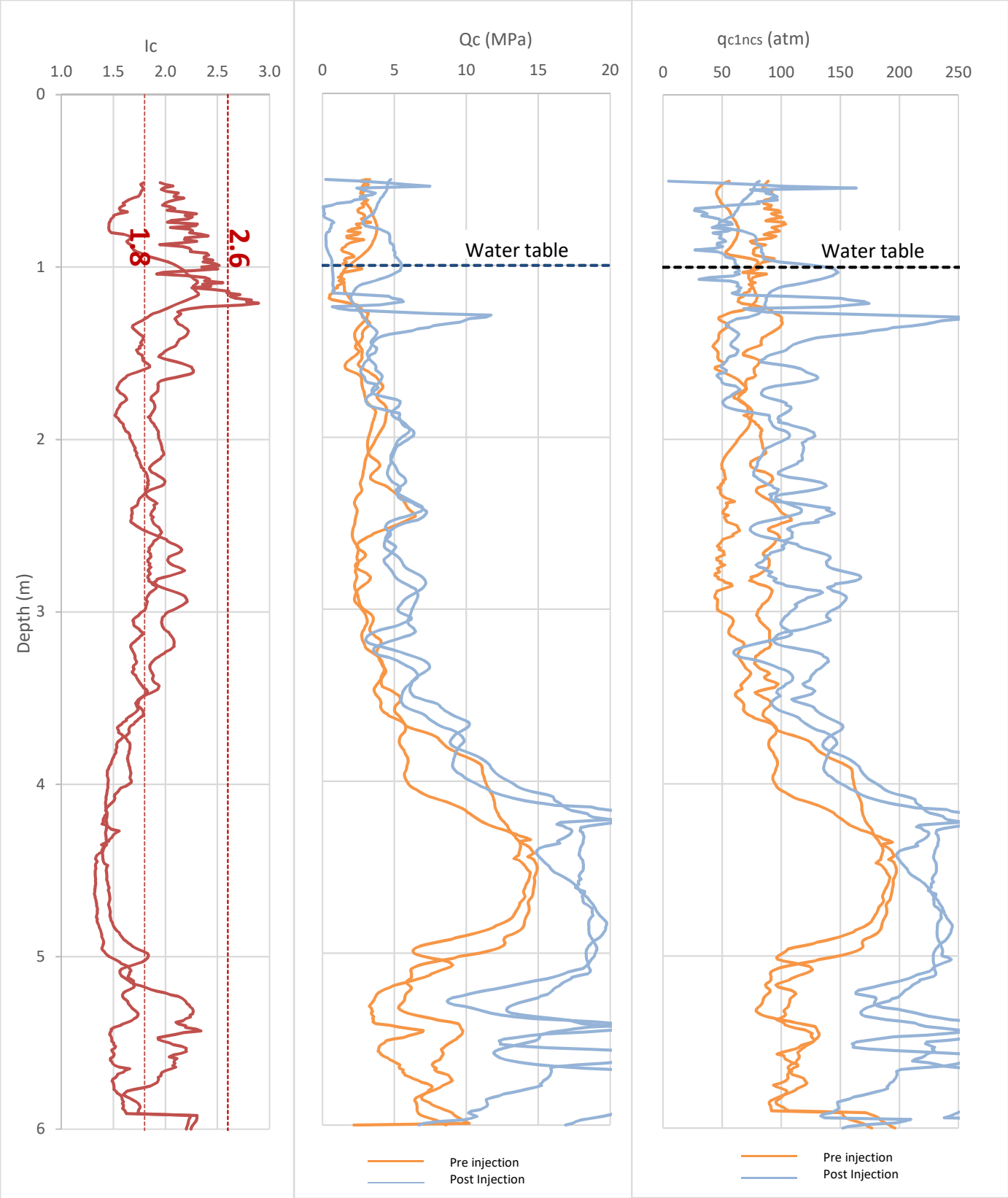
GEOTECH CONSULTING LTD

Project:	Red Zone trials Resin Injection	Loc No:	**Production Panel 1**
Client:	Mainmark Ground Engineering Ltd	Job No:	4286 Site 3



Average pre-inj K_D:	8	Average pre-inj M:	56 MPa	(2-6m depth)
Average post-inj K_D:	12	Average post-inj M:	131 MPa	(2-6m depth)

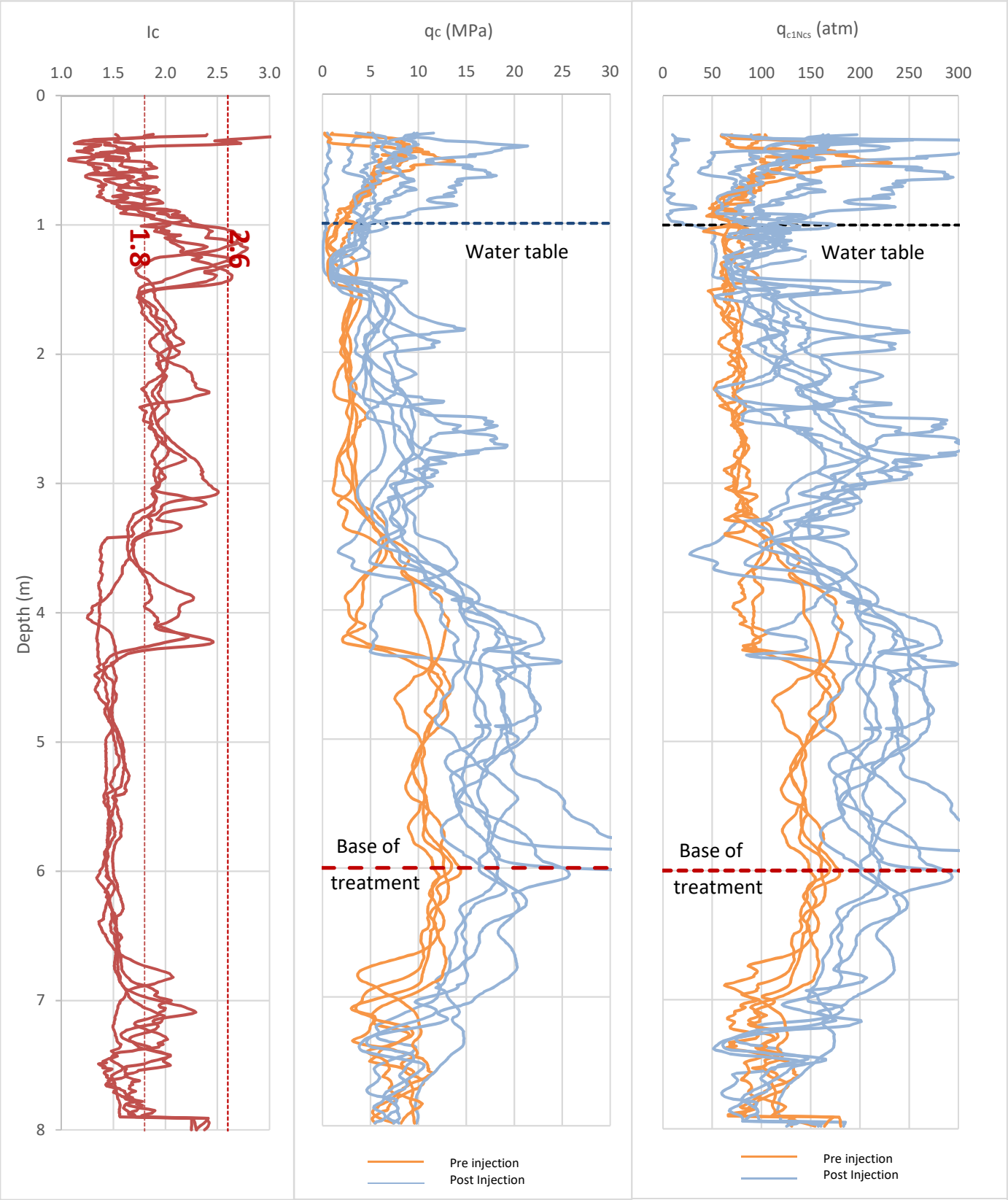
GEOTECH	CONE PENETROMETER TEST DATA			
	GEOTECH CONSULTING LTD			
	Project:	Red Zone Trials Polyurethane Injection	Loc No:	**Spacing Panel 1**
	Client:	Mainmark Ground Engineering (NZ) Ltd	Job No:	4286 Site 3



Average pre Qc:	6	MPa	Average pre qc1ncs:	103	atm
Average post Qc:	10	MPa	Average post qc1ncs:	155	atm

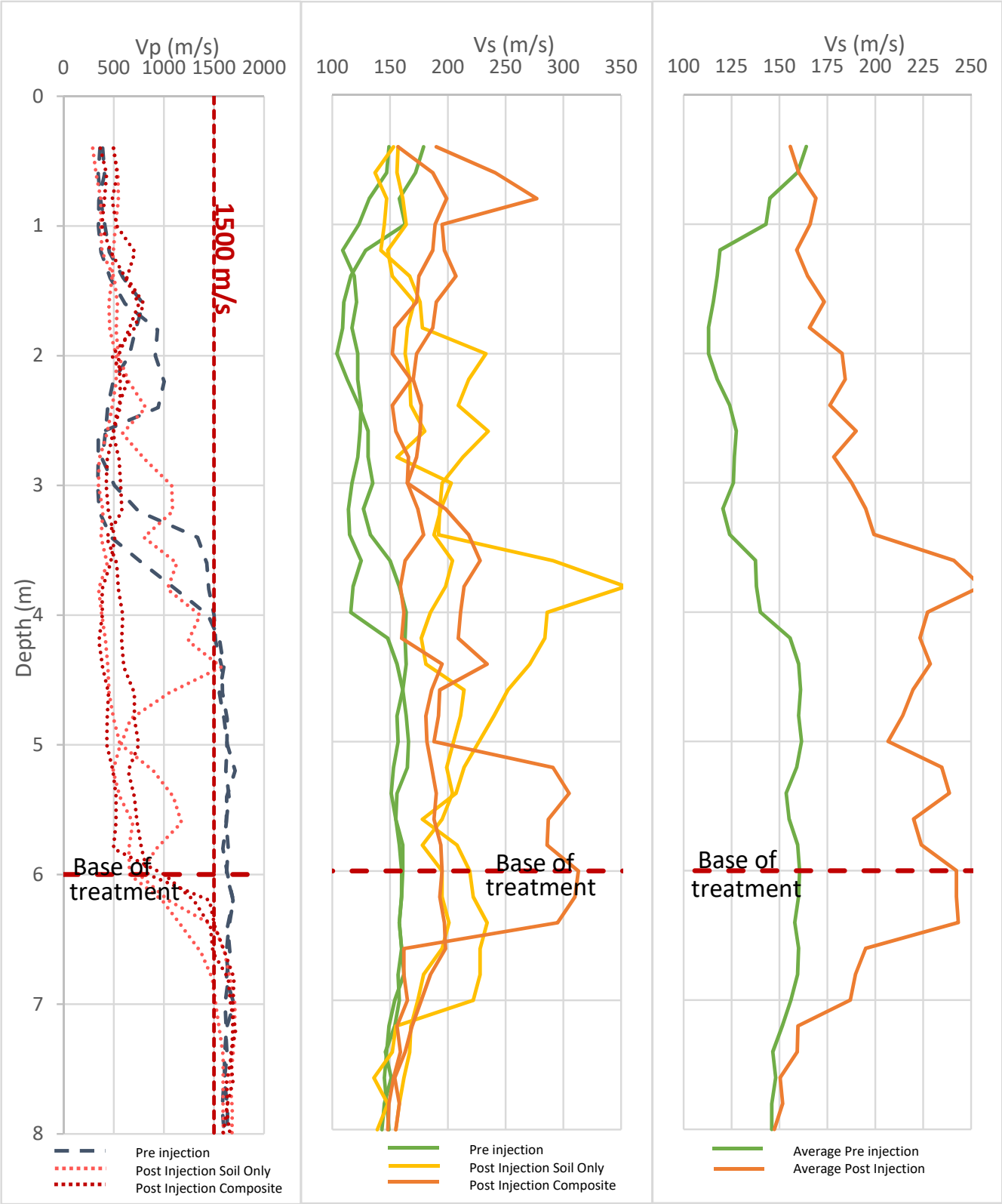
Appendix F
Test Results Site 4

GEOTECH	CONE PENETROMETER TEST DATA			
	GEOTECH CONSULTING LTD			
	Project:	Red Zone Trials Polyurethane Injection	Loc No:	**Production Panel 1**
	Client:	Mainmark Ground Engineering (NZ) Ltd	Job No:	4286 Site 4



Average pre q_c:	6	MPa	Average pre q_{c1Ncs}:	107	atm
Average post q_c:	11	MPa	Average post q_{c1Ncs}:	175	atm

GEOTECH	GEOPHYSICAL TESTING DATA			
	GEOTECH CONSULTING LTD			
	Project:	Red Zone trials Resin Injection	Loc No:	**Production Panel 1**
	Client:	Mainmark Ground Engineering Ltd	Job No:	4286 Site 4

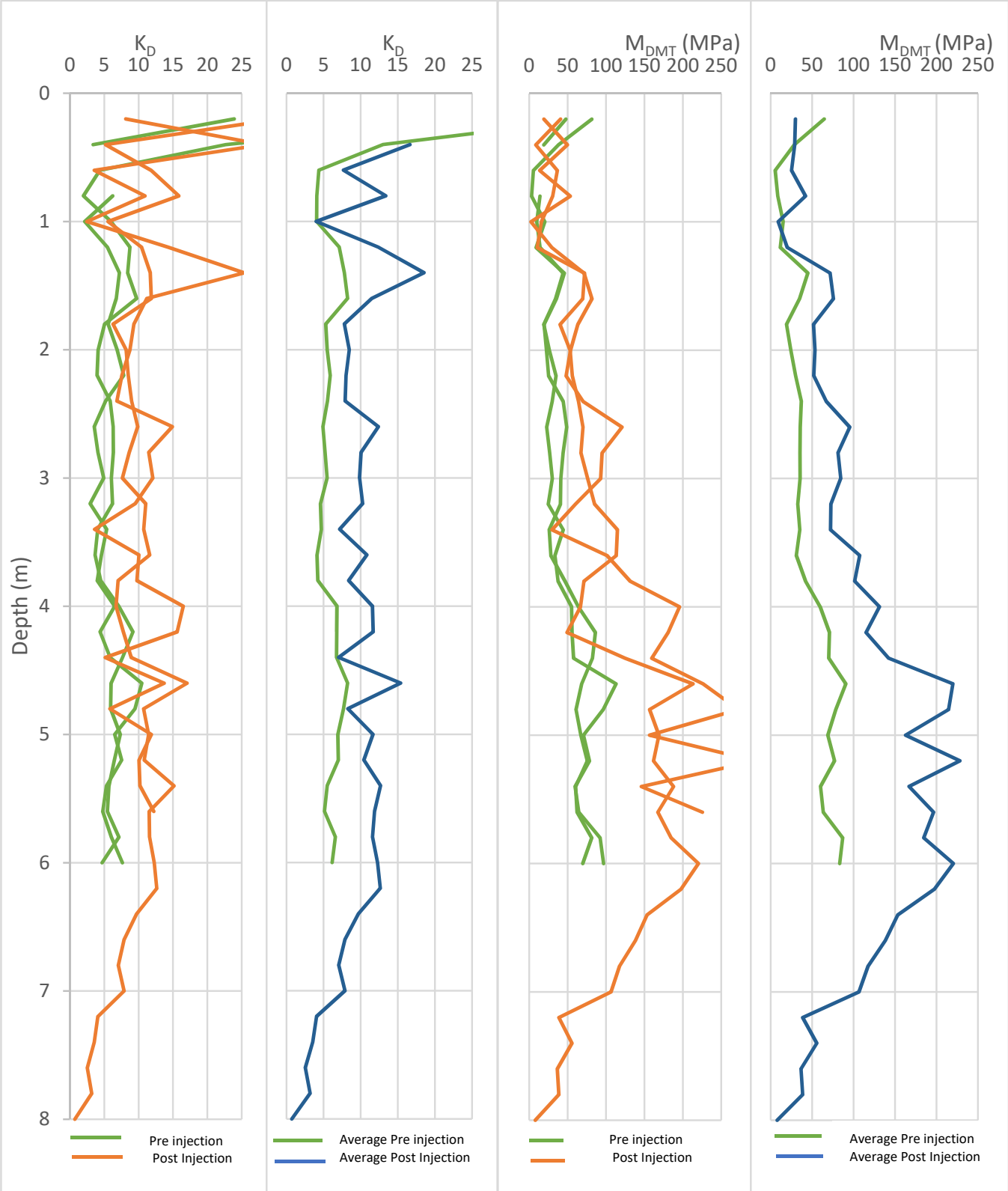


Average pre-inj:	138 m/s	Average post (composite):	194 m/s
Average post (soil):	202 m/s	Average post (overall):	198 m/s

DILATOMETER TESTING DATA

GEOTECH CONSULTING LTD

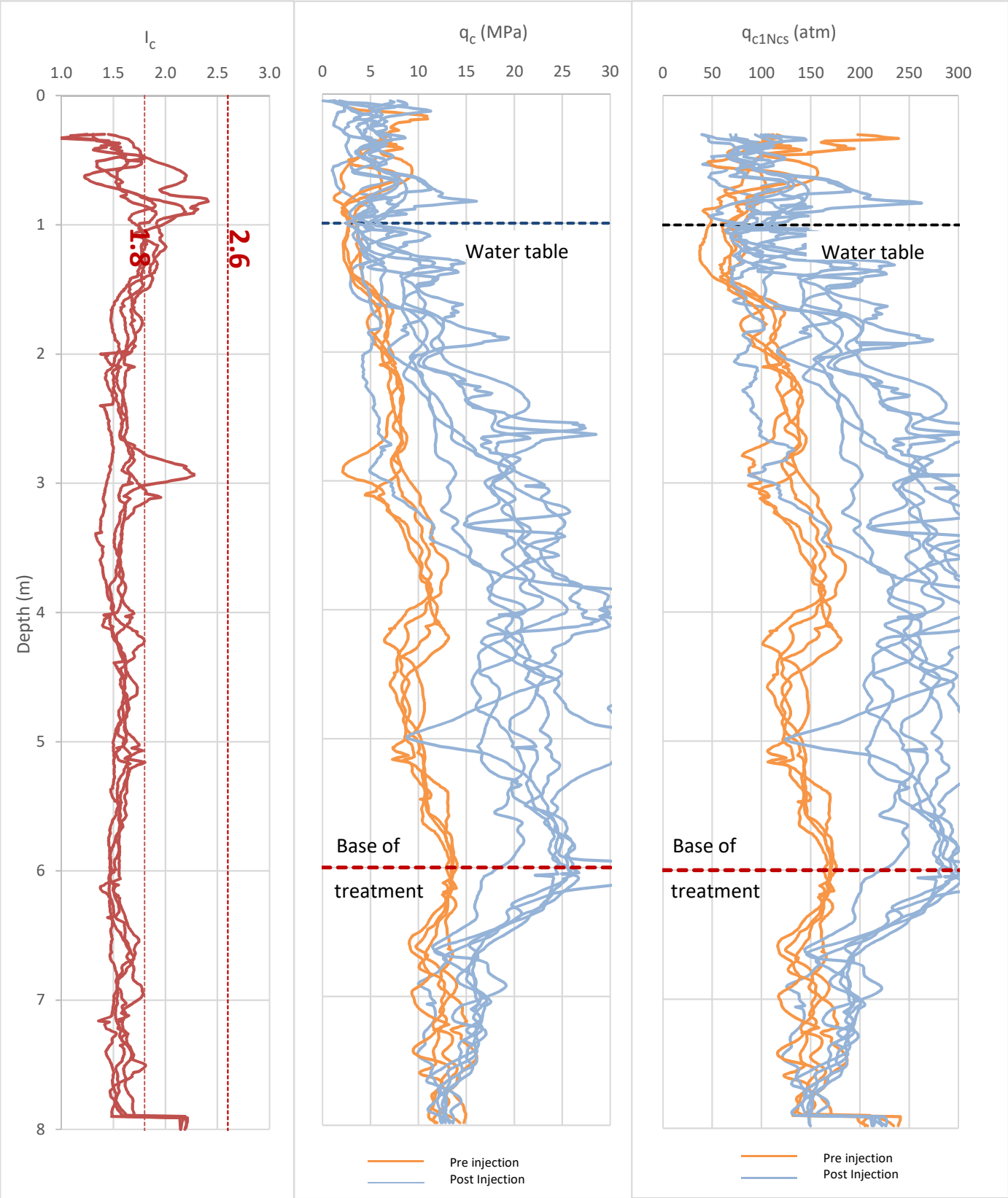
Project:	Red Zone trials Resin Injection	Loc No:	**Production Panel 1**
Client:	Mainmark Ground Engineering Ltd	Job No:	4286 Site 4



Average pre-inj K_D:	6	Average pre-inj M:	49 MPa	(1-6m depth)
Average post-inj K_D:	10	Average post-inj M:	115 MPa	(1-6m depth)

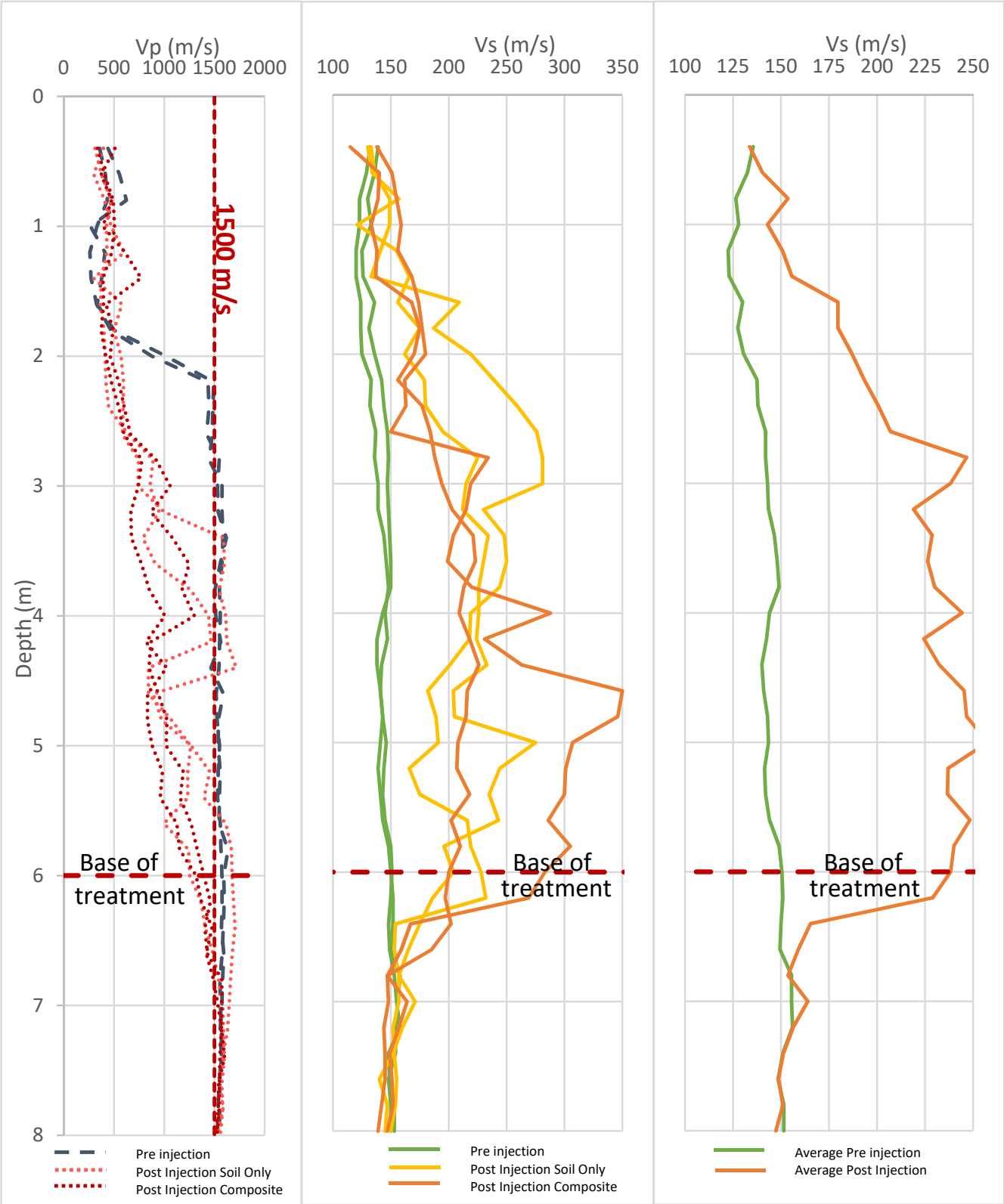
Appendix G
Test Results Site 6

GEOTECH	CONE PENETROMETER TEST DATA			
	GEOTECH CONSULTING LTD			
	Project:	Red Zone Trials Polyurethane Injection	Loc No:	**Production Panel 1**
	Client:	Mainmark Ground Engineering (NZ) Ltd	Job No:	4286 Site 6



Average pre q_c:	9	MPa	Average pre q_{c1Ncs}:	128	atm
Average post q_c:	17	MPa	Average post q_{c1Ncs}:	225	atm

GEOTECH	GEOPHYSICAL TESTING DATA			
	GEOTECH CONSULTING LTD			
	Project:	Red Zone trials Resin Injection	Loc No:	**Production Panel 1**
	Client:	Mainmark Ground Engineering Ltd	Job No:	4286 Site 6

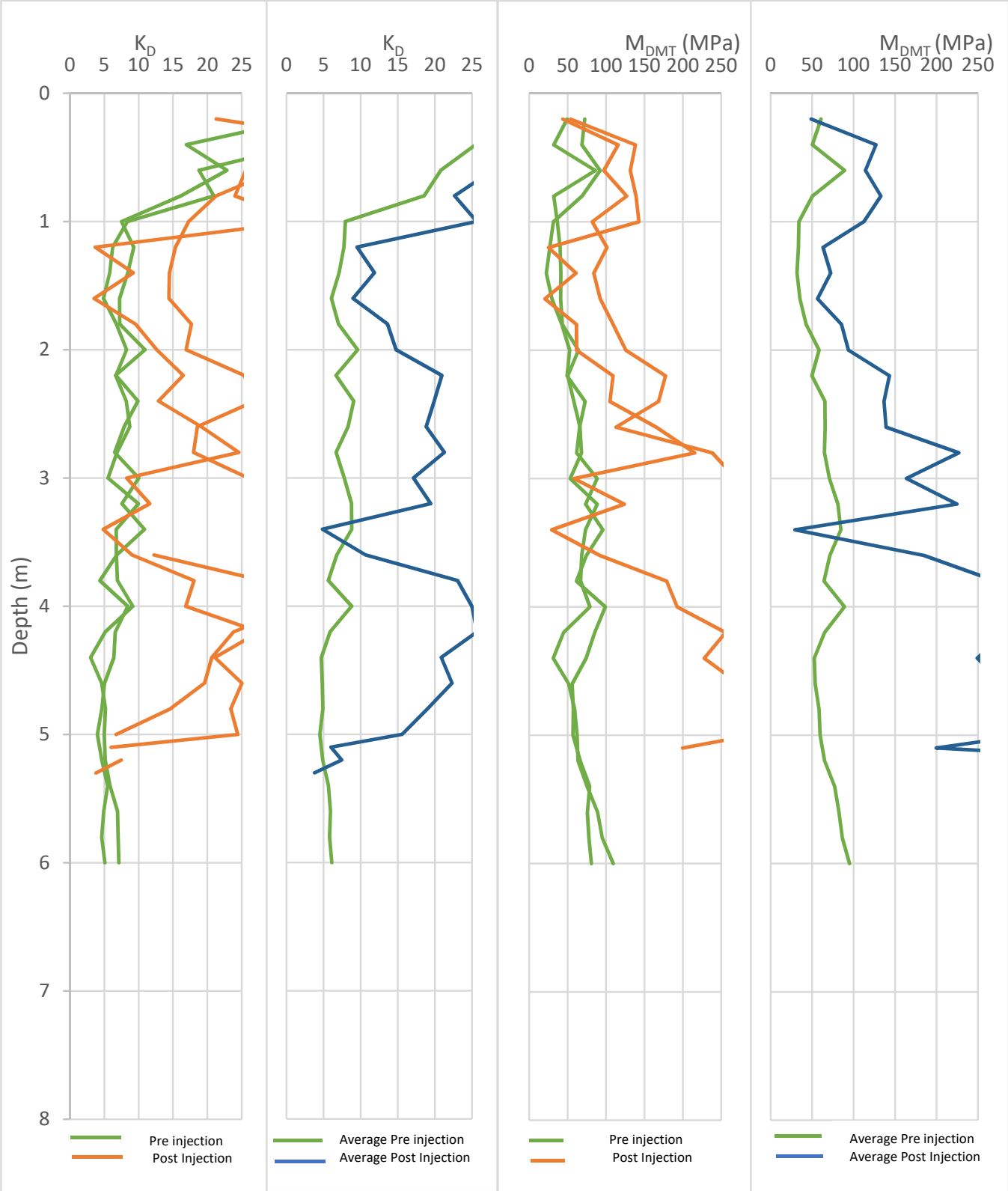


Average pre-inj:	140 m/s	Average post (composite):	212 m/s
Average post (soil):	209 m/s	Average post (overall):	211 m/s

DILATOMETER TESTING DATA

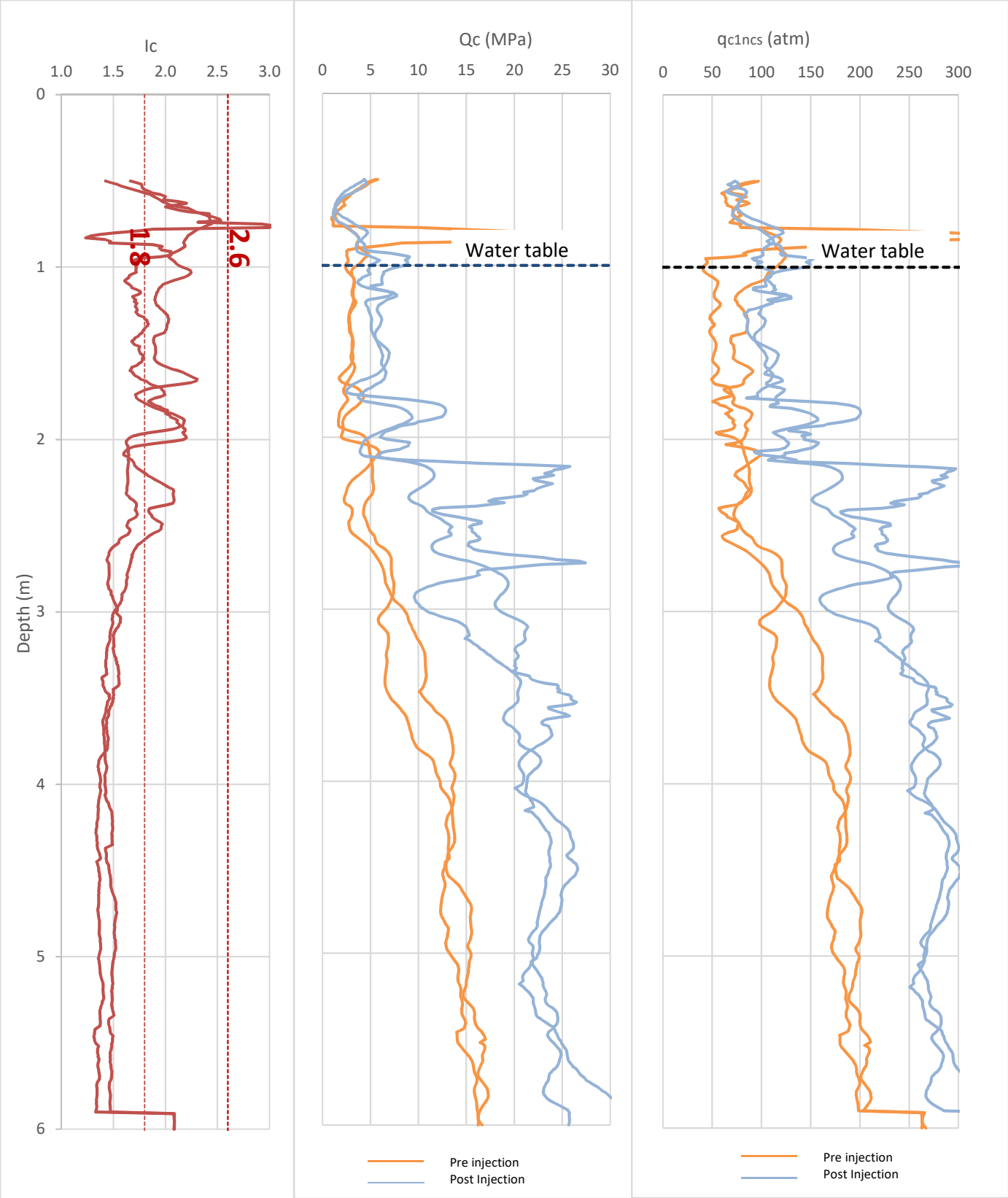
GEOTECH CONSULTING LTD

Project:	Red Zone trials Resin Injection	Loc No:	**Production Panel 1**
Client:	Mainmark Ground Engineering Ltd	Job No:	4286 Site 6



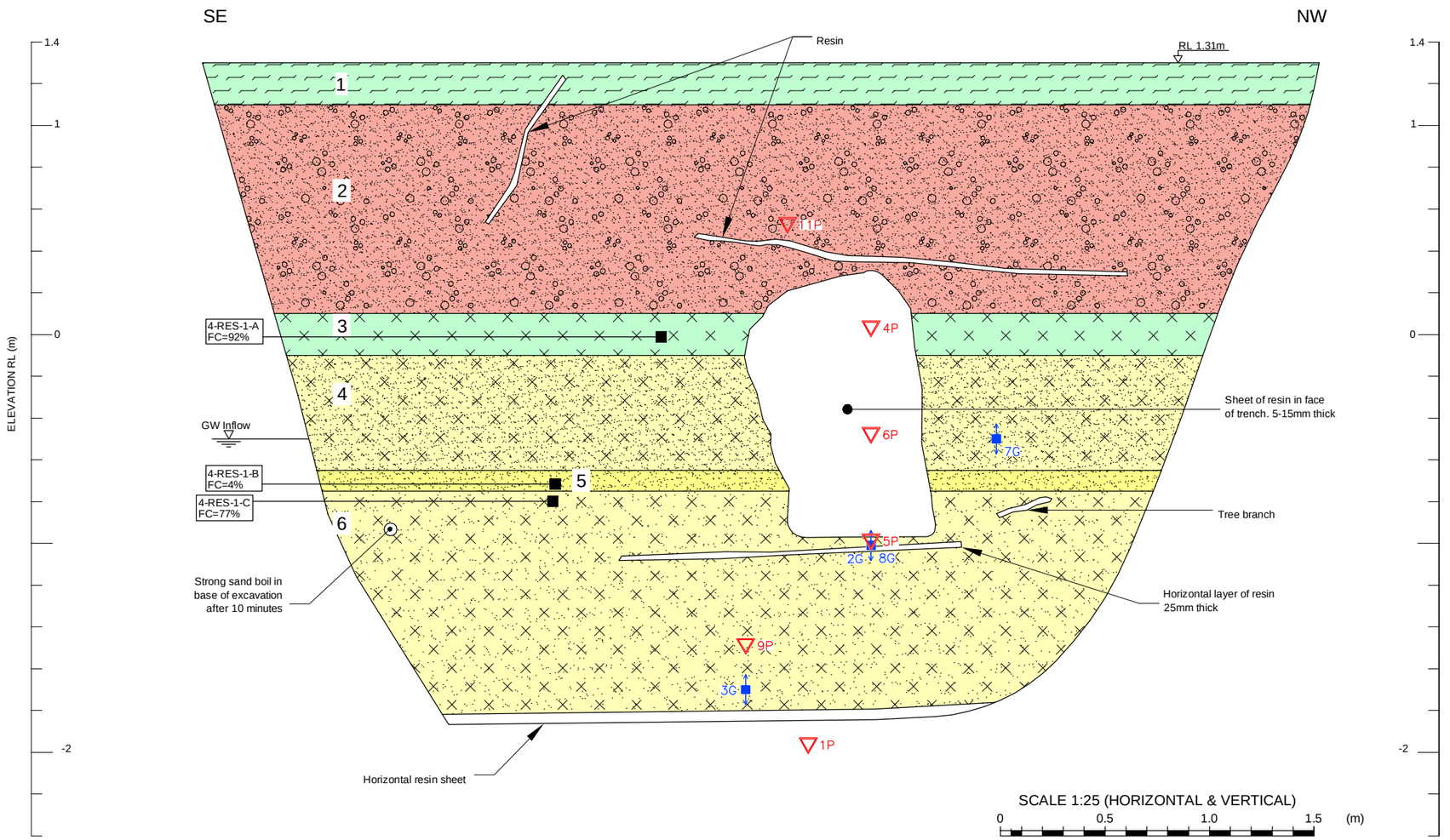
Average pre-inj K_D:	7	Average pre-inj M:	59 MPa	(1-5m depth)
Average post-inj K_D:	18	Average post-inj M:	177 MPa	(1-5m depth)

GEOTECH	CONE PENETROMETER TEST DATA			
	GEOTECH CONSULTING LTD			
	Project:	Red Zone Trials Polyurethane Injection	Loc No:	**Spacing Panel 1**
	Client:	Mainmark Ground Engineering (NZ) Ltd	Job No:	4286 Site 6



Average pre Qc:	9	MPa	Average pre qc1ncs:	139	atm
Average post Qc:	18	MPa	Average post qc1ncs:	229	atm

Appendix H
Test Results EQC Trials



TRENCH LOG LEGEND

- 1 — Topsoil. (FILL). **(USCS - organic silt, OL)**
- 2 — Sandy fine to coarse GRAVEL; brown. Gravel, rounded. (FILL). **(USCS - poorly graded gravel with silt and sand, GP-GM)**
- 3 — SILT with minor sand; grey, homogeneous. Very stiff; moist; organic odour. **(USCS - silt with sand, OH)**
- 4 — Silty fine SAND with minor organics; brownish grey. organics, amorphous. **(USCS - silty sand, SM)**
- 5 — Fine SAND with trace silt and minor organics. Loose. **(USCS - poorly graded sand, SP)**
- 6 — Silty fine SAND with some organics; grey, homogeneous. Loose; organics, fibrous. **(USCS - silty sand, SM)**
- Soil sample
- Pore Pressure Transducer
- 2D Geophone

LOCATION PLAN LEGEND

- Logged face of trench
- RI01 — Installation location
- Direction of section through trench
- Location of control panel

Resin sheet in side of trench.

Stratigraphy and resin in trench wall.

LOCATION PLAN - 1134 AVONSIDE DRIVE

REF: Aerial Imagery dated February 24, 2011 was supplied by New Zealand Aerial Mapping Ltd.

N.T.S

NOTES:

1. Trench orientated at 135°.
2. FC based on 0.075mm sieve aperture.
3. FC = Fines Content.
4. FC rounded to nearest 1%.
5. Logged to NZGS Guidelines.
6. Horizontal location of Pore Pressure Transducers and 2D Geophones is approximate.
7. RL Elevation in terms of Lyttelton Vertical Datum.

ORIGINAL IN COLOUR

LOGGED BY: G Halliday & R Hunter 23/10/2013

EQC EARTHQUAKE COMMISSION Kōmihana Rūwhenua

DRAWN	HSJ	May.14
DRAFTING CHECKED	PWQ	May.14
APPROVED	SVB	May.14
CADFILE : \\S4 - RES - 1.dwg		
SCALES (AT A3 SIZE) AS SHOWN		
TT PROJECT No. 52020.023		

Ground Improvement Trials

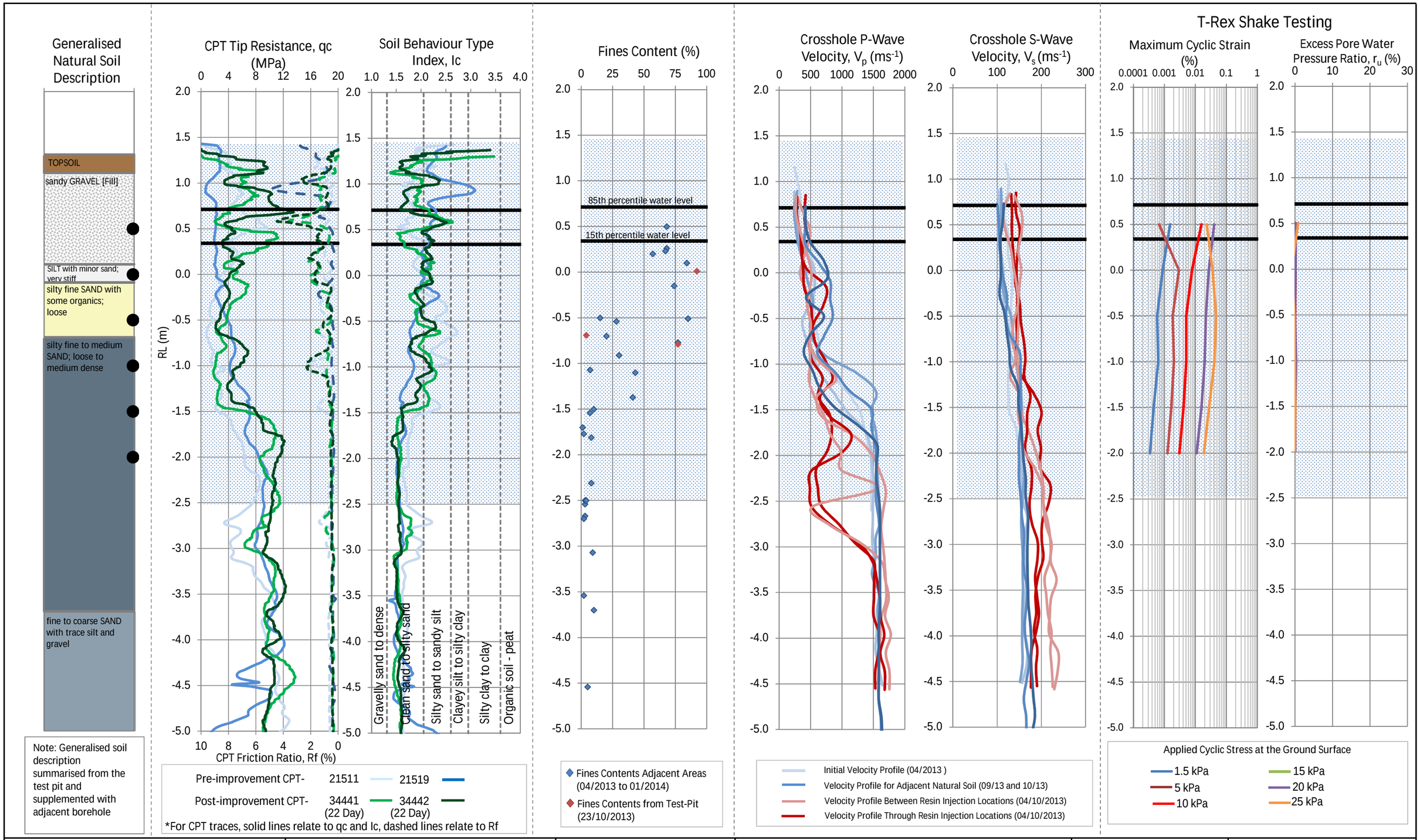
Site 4 (Avondale)

Resin Injection Panel 1 (4-RES-1)

Trench Log

Figure E5.13.2

REV. 0



LEGEND
- 85th and 15th percentile groundwater levels
- Pore Pressure Transducer Installation Elevation
- Depth of Improvement (0 - 4m)

Notes:
Natural Soil Crosshole Results Obtained from:
VsVp 36460 (Initial Site Crosshole Test), 4-NS-1 and 4-CFA-1
Adjacent Lab Test Data Obtained from:
BH 21495, BH 34456, BH 39758 4-NS-1, 4-RIC-1, 4-RAP-1 4-DTP-1

EQC
EARTHQUAKE COMMISSION
Kōmihana Rūwhenua

Ground Improvement Trials
Site 4 (Avondale)
Resin Injection Panel 1 (4-RES-1)
Summary of Testing Results

Location: Avondale
Job No.: 52020.023
Version No.: 1.8

Figure E5.13.3

Date: 15/04/2014
Analysed by: JUL
Checked by: PWQ