Liquefaction Mitigation beneath Existing Structures Using Polyurethane Grout Injection

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ABSTRACT: While ground improvement of bare land for liquefaction mitigation purposes has become more commonplace in New Zealand since the 2010 – 2011 Canterbury Earthquake Sequence, relatively few practical methods are currently available for liquefaction mitigation beneath existing structures. One such method that is showing promise is the injection of expanding polyurethane grout. For a number of decades this technology has been used worldwide for the relevelling of structures, however this process may be able to be used to densify the ground and thus increase liquefaction resistance (i.e. increase the relative density and therefore the cyclic resistance ratio of the soils), as well as improve the composite stiffness of the ground. This paper describes some early test results from the recent use of expanding polyurethane grout injection for liquefaction mitigation purposes beneath a set of existing commercial structures in Christchurch.

1 INTRODUCTION

Since the widespread occurrence of seismically induced soil liquefaction and consequent foundation and building damage during the 2010 – 2011 Canterbury Earthquake Sequence, much attention has been focussed in Christchurch on ground improvement techniques. Most of the commonly available methods of ground improvement require the proposed treatment area to be free of structures, in order to be viable. These include stone column installation, ‘excavate and recompact / replace’, dynamic compaction, densification by driven piles, and cement stabilisation methods.

Methods that can be used under existing buildings are less readily available, despite there being a demand for such technologies. As part of the Canterbury earthquake recovery, a number of buildings are being restored back to (and beyond) pre-earthquake condition by increasing the liquefaction resistance of the underlying soils, for example by the use of jet grouting. Additionally, much of the country’s commercial building stock is now subject to the Detailed Seismic Assessment (DSA) process. This will result in a significant number of buildings being targeted for upgrading, either for regulatory reasons (to meet minimum required standards) or for commercially driven reasons (i.e. to increase a buildings earthquake resilience and thereby make a building more attractive to potential tenants). On sites where the structural response of the building is potentially affected by liquefaction of the underlying soil layers, an in-situ method of liquefaction mitigation that does not require the removal of the structure will be an important consideration in building upgrade schemes.

In Christchurch, Jet grouting is currently the most common method in use for this situation, and compaction grouting using LMG (low mobility grout) has been used in some circumstances. The EQC ground improvement trials also pioneered a technique of HSM (horizontally soil mixed) beams which offers some increased resilience at shallow depths under existing buildings (Hunter et al., 2015; Wansbone and van Ballegooy, 2015). Permeation grouting can be used in clean coarse sands, however this has not been utilised in Christchurch due to the (generally) finer-grained nature of the soils there (making permeation difficult), as well as the high cost of this technology. Further methods being researched at present include de-saturation and calcification by biological means, (Kavazanjian et al. 2015), and densification by polyurethane grout injection.

This paper presents some early field trial results from a liquefaction mitigation project beneath a commercial structure that is underway in Christchurch, using injection at depth of an expanding polyurethane grout.

2 THE PROJECT

Three adjoining 'big box' 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) are currently being relevelled, repaired and upgraded. The relevelling is being carried out using 'JOG' (a computer controlled cement based micro-injection process) and polyurethane injection methods, followed by structural strengthening. The buildings in this case are being 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 the upper 4m to 7m of the soil profile). The aim is to reduce liquefaction-induced damaging differential settlements. The liquefaction mitigation works are being carried out as a 'Design and Construct' project by Mainmark Ground Engineering (NZ) Ltd, in advance of the other works, using pressure injection of an expanding polyurethane grout mix into the ground at depth, to densify the underlying soils. This is achieved by drilling holes through the concrete floor slab or surface fill, inserting grout tubes into the ground to the desired depth, and introducing the polyurethane mix at the grout tube tip as the tubes are withdrawn.

The buildings' load-bearing foundation areas are 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 floor areas of 4 metres, and 7 metres depth under the wall foundations.

A series of test panels (see Figure 1 below) were carried out as part of the works, in order 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 one panel was installed adjacent to the external (load bearing) wall of the same building. 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 internal (shop floor) test panel was installed over five night-time occupations. The production works will be carried out on the same basis, with minimised interruption to the continued occupation and operation of the buildings.

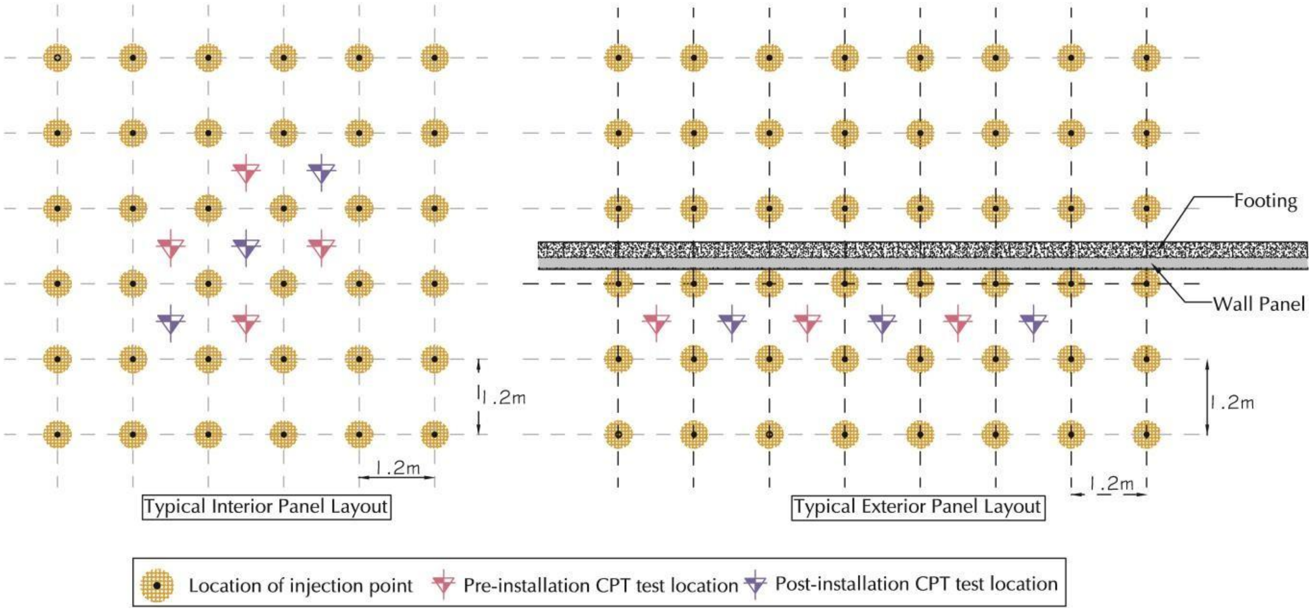

Figure 1 - Typical Test Panel Layouts (Plan View)

3 SOIL CONDITIONS

Soil conditions are somewhat variable across the project site, consisting mainly of surficial fill up to 1m thick, overlying interbedded loose sand to silty sand (in the northern part of the site) or silty sands and silts (in the southern part of the site) to about 6m depth. A loose silt to clayey silt then extends to about 8m depth, and loose sandy silts or silts to 14 to 16m depth. Below this are medium dense to dense sand layers to 20m, where a dense gravelly layer (the "Riccarton Gravels") is struck. In the north-western part of the site a relatively dense sandy gravel to gravelly sand unit extends into this general profile, between 4m and 7 to 10m depth. The water table is at about 1m depth.

The soil conditions are noticeably siltier within the treatment zone at the internal test panel location than at the locations of the (more northern) external test panels. This is consistent with the site having siltier conditions in its southern part. Within the area of the test panels in the north, the soil conditions also varied, with gravel intrusions at depth in some of the test panels.

Under design levels of ground shaking, the upper 10 to 14m of the soil profile is predicted to liquefy (based on the Boulanger & Idriss (2014) simplified assessment method), with a crust thickness of only about 1 to 1.5 metres.

4 SOIL IMPROVEMENT MECHANISM

The liquefaction mitigation design relied solely on densification of the soil (although other effects such as improvement in stiffness and cementation may also be present), from the injection of an aggressively expanding polyurethane product. Once injected at any particular depth, the low viscosity polyurethane both permeates the soil to some extent and also penetrates under pressure along planes of weaknesses within the soil profile. The polyurethane mix reacts soon after injection, rapidly expanding to many times it original volume. The expansion of the injected material results in compaction of the adjacent soils, as additional material is introduced into a relatively constant soil volume. This is a similar effect to the 'Area Replacement Ratio' (ARR) enhancement provided by vibro replacement stone columns or compaction grouting (for example). These processes can result in some soil heave at the ground surface. (This process is also often used at shallower depths to lift and relevel buildings).

It should be noted that the expanding polyurethane injection process does not produce regular columns or spherical nodules of materials distributed down the vertical injection line. Instead it typically results in a 'veining' of expanded material distributed through the soil mass as dykes, sills or networks of sheets, typically tens of millimetres thick (EQC).

While the design of the mitigation works for this project relied solely on soil densification, earlier preliminary trials of this technology in 2013 during the EQC Ground Improvement Trials (EQC) have shown two other potential improvement mechanisms. Direct push cross-hole geophysical testing has shown an increase in the composite shear stiffness of soils that have been treated in this manner (an increase in shear stiffness decreases the liquefaction potential of a soil), and also a desaturation effect (which also decreases liquefaction potential). The longevity of the desaturation effect is currently unknown, so this potential effect is not relied upon in the design process. The use of direct push cross-hole geophysical testing is however available as a back-up verification method.

5 SOIL IMPROVEMENT VERIFICATION

The verification process for the project is based on measuring the post-injection CPT cone resistance at the midpoint between injection locations one month after the polyurethane injection process has been carried out, at a set sampling rate across the works area. The CPT-based parameter q_{c1Ncs} (the CPT cone resistance normalised and corrected to one atmosphere confining pressure, and also corrected for fines content) is the direct input into the calculation of the cyclic resistance ratio (CRR) of a soil using the Boulanger & Idriss (2014) assessment method, and this parameter was therefore used as the basis of performance verification for the project. Densification of a soil from horizontal stresses can affect the apparent fines content of a soil inferred from CPT data - therefore it is important when interpreting post-improvement CPT data to use fines contents derived from pre-improvement CPT data (Nguyen et al. 2014), or from laboratory soils test data. For this reason a series of pre-improvement CPT tests were carried out, as close as possible to the location of the proposed post-improvement tests. These pre-improvement CPT tests also allow assessment of the degree of improvement achieved by this process.

In a number of cases for the pre-production trial panels the CPT probe refused on dense materials at between 4 and 8 metres depth. In these instances pre-drilling was carried out to allow further advancement of the CPT, and limited SPT testing was carried out in the pre-drill zones.

6 SOIL IMPROVEMENT RESULTS

The depth of injection, soil profiles, amount and type of material injected, as well as injection spacing and surface overburden varies between the test panels. These variations are not fully quantified in this paper, which primarily seeks to examine whether or not this technology is viable for increasing soil densities - however some of the variations are detailed in Table 1.

Table 1. 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)
EQC	0-4m	1.2m	1.6	45%	45%	N/A**
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 ** Reliable Ic data not available

Presented below are graphical representations of the improvement results from both the five test panels at the project site, as well as the earlier single panel from the EQC ground improvement trials. Figures 2 to 7 (Set 1) show results in terms of the more well-known CPT tip resistance (Qc), and Figures 8 to 12 (Set 2) are presented in terms of q_{c1ncs}. This second presentation is a truer reflection of the improvements gained, as it effectively account for the soil behaviour type (Ic).

In these figures the individual 'before and after' CPT traces show considerable variation, although increases in CPT tip resistances are generally discernible. Once the results are collapsed into average 'before and after' traces, the improvement effect is clearer.

In all cases the post installation results show overall improved CPT tip resistances (and q_{c1ncs}) in the treated soil zones, and therefore improved soil densities and increased resistance to liquefaction. The amount of improvement varies somewhat between the test panels. This is influenced to some degree by soil conditions, which vary considerably between the test panels both in pre-treatment density as well as soil behaviour type (Ic). The presence of gravel intrusions has led to reduced certainties in the comparison of test results between test panels. Variations due to the amount of material inserted at each injection point, the spacing of injection points, as well as confinement are less discernible, as pattern of increasing penetration resistance with increasing material injection or depth does not appear immediately obvious. However when only the upper 4m of the soil profile is examined (where the more reliable data is found due to soil conditions there being more uniform across the test panels), some differences are more apparent (refer Table 1). In some cases the improvement effect appears to extend below the treatment depth, but in other cases this has not occurred.

Set 1 – Plots of CPT Cone Resistance Qc

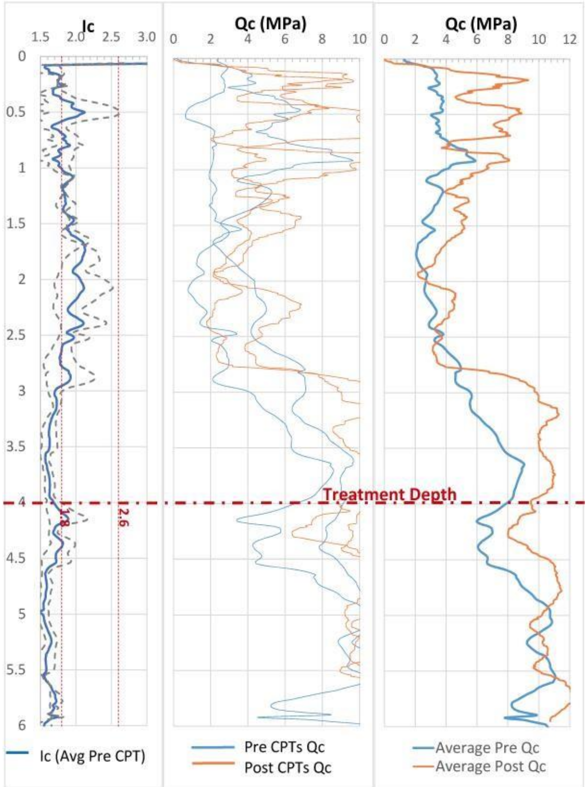

Figure 2 - EQC 2013 Ground Improvement Trial

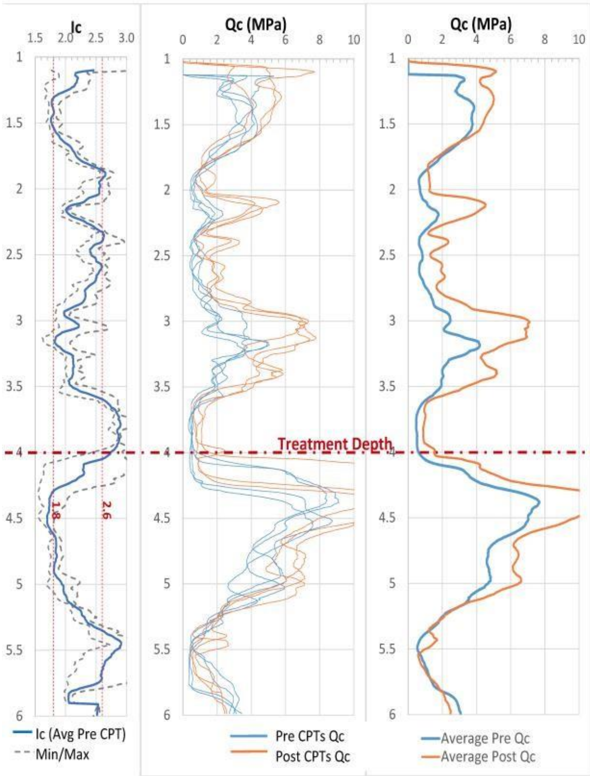

Figure 3 - Interior Test Panel 1

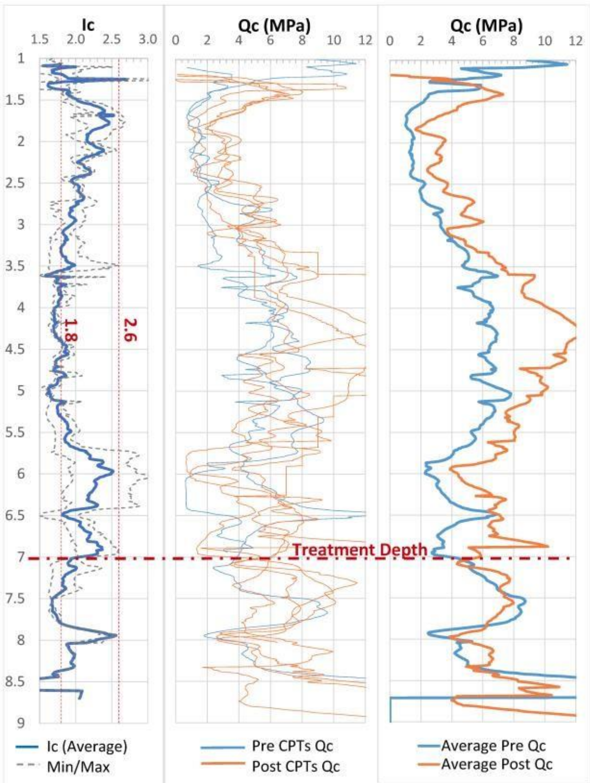

Figure 4 - Exterior Test Panel 1(with data from SPT)

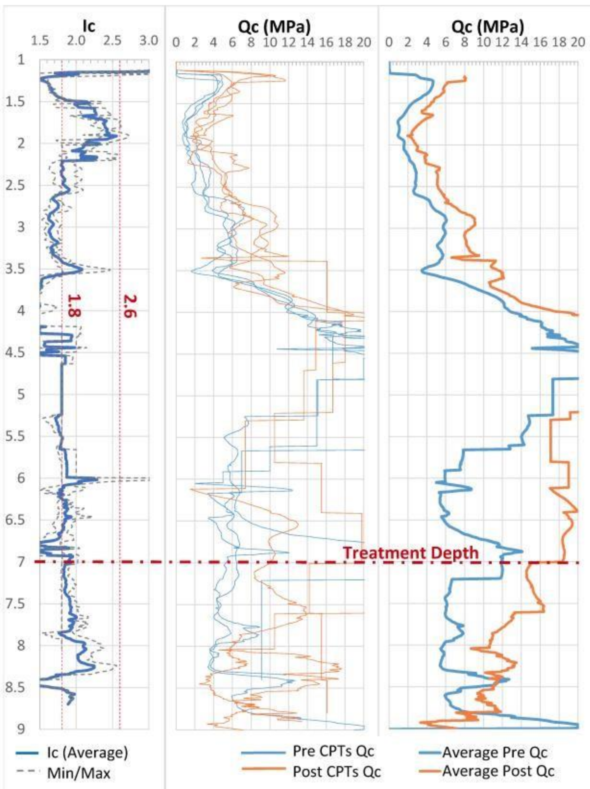

Figure 5 - Exterior Test Panel 2 (with data from SPT)

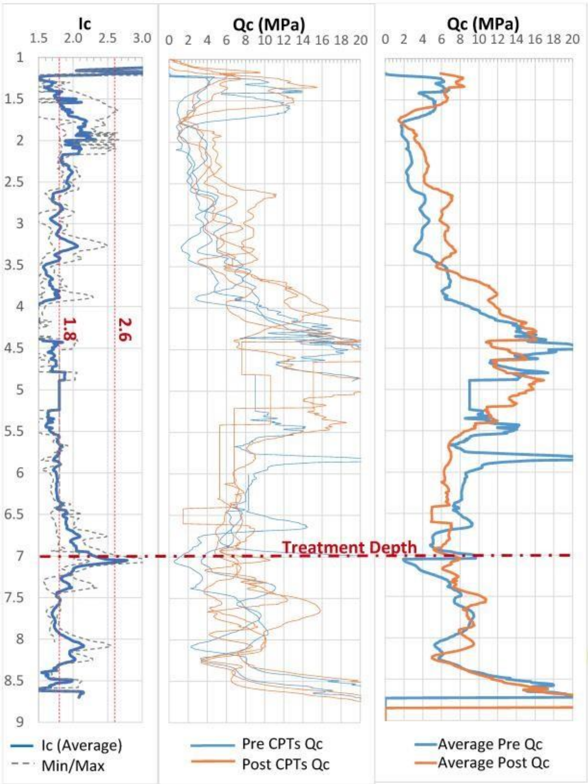

Figure 6 - Exterior Test Panel 3 (with data from SPT)

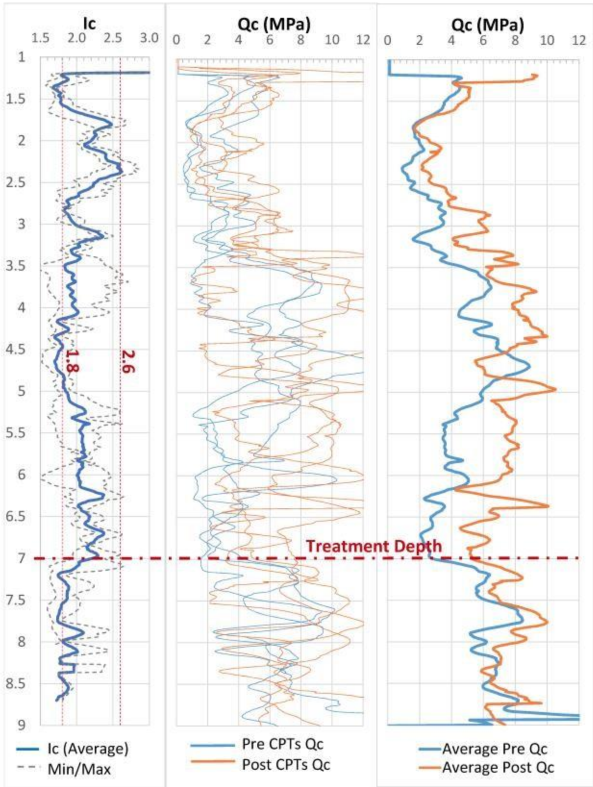

Figure 7 - Exterior Test Panel 4

Set 2 – Plots of q_{c1ncs}

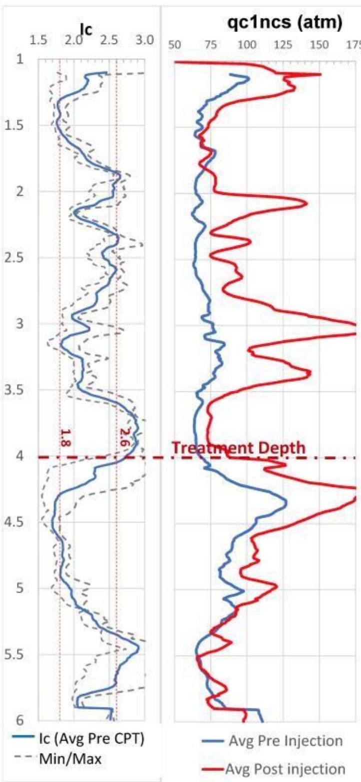

Figure 8 - Interior Test Panel 1

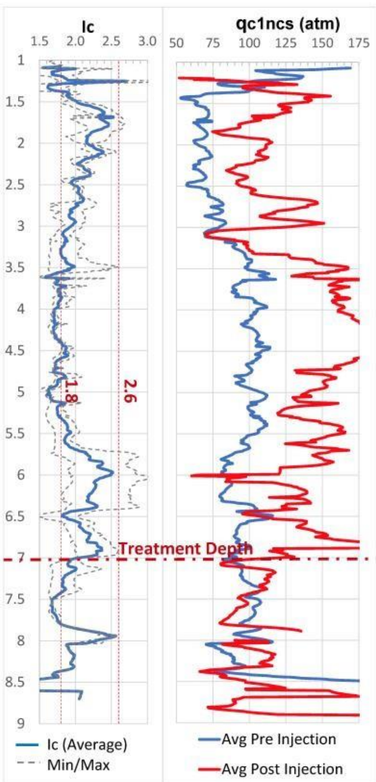

Figure 9 - Exterior Test Panel 1

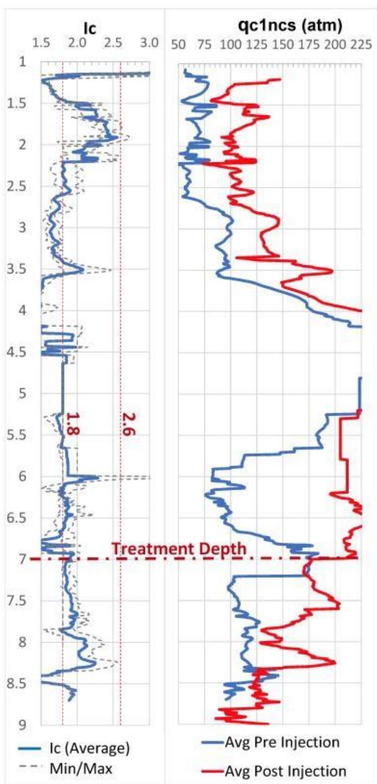

Figure 10 - Exterior Test Panel 2

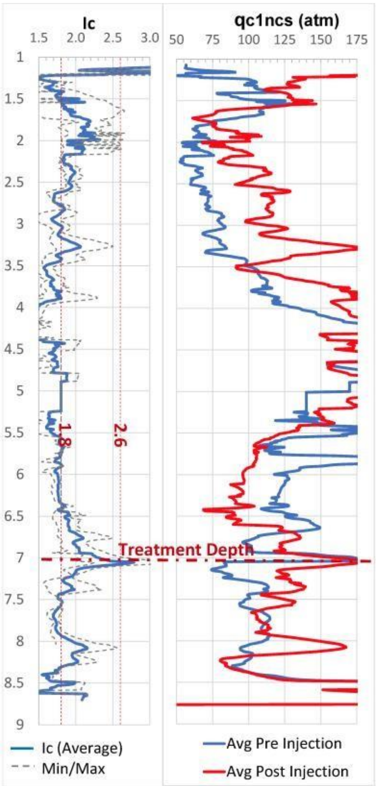

Figure 11 - Exterior Test Panel 3

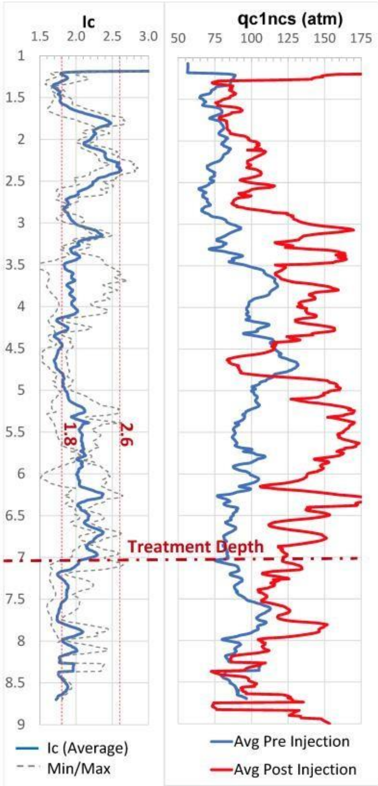

Figure 12 - Exterior Test Panel 4

In one location (Exterior Test Panel 3) the post injection data seems to indicate a reduction in cone resistance at some depths, however this may be due to the conversion from SPT data to CPT data at those depths (where a gravel intrusion complicates matters), and also SPT data from within gravel deposits, both of which can be unreliable. (For comparison purposes, SPT data was converted to approximately equivalent CPT data using an 'equivalent CRR' process, and tends to show up in the plots in Figures 4 -12 as straight lines in the data).

An assessment of average free-field liquefaction-induced settlements using the method of Zhang et al. (2002) were made for the test panel locations, in the upper 10m of the soil profile. Reductions in both settlements, as well as Liquefaction Severity Number (LSN) (van Ballegooy et al., 2014) at ULS levels of shaking (M7.5, 0.35g) of approximately 35 -40% were calculated, and 60 – 70% reductions were assessed for the SLS case (M6, 0.19g).

By inspection of Figures 2 – 12 there appears to be a loose trend of increasing effectiveness with lower Ic values, which is common with many ground improvement methods. However even with soils approaching Ic values of 2.6 - i.e. clay like behaviour (Robertson and Wride 1997) beyond which liquefaction is generally considered unlikely to occur, improvements in CPT tip resistance were still observed.

In all cases some soil heave, or lifting of the building floor, was observed – this tended to occur while injecting in the upper two metres of the soil profile. (Typically this uplift partially subsided after a period of a several hours, likely due to redistribution of pore pressures). For a building that needs to be lifted, or is not adversely affected by an increase in finished floor level, this may not be a major consideration, but for other buildings this effect would need to be considered.

Due to very tight project time constraints, soils laboratory test data was not available at the time of the design for the ground improvement works. Under normal circumstances it is typically recommended that laboratory testing is carried out to determine fines contents and plasticity properties for the soils to be treated, in order to refine the liquefaction triggering assessment process. At the time of this paper going to press, some laboratory testing results for the site were becoming available, tending to indicate that the liquefaction hazard in the southern part of the building area (where the siltier soils are generally located) is less than that indicted by the CPT data (i.e. when assuming the default fines content correlations from the CPT Ic values). This will likely result in a reduction in the required degree or extent of soil treatment.

7 CONCLUSIONS

These preliminary results have demonstrated that soil improvement using injected expanding polyurethane grout is viable, and worthy of further research. The project has also shown that this technology can be applied successfully beneath existing buildings, without necessarily interrupting the use of those building. The trial panel data indicates that improvement can occur across a wide range of liquefiable soils, with increasing improvement often being noted at lower Ic values (typically less than 2). While the majority of the results presented in this paper come from the application of the expanding urethane grout technology on a commercial project (with all the constraints associated with such an environment), a research project is underway where similar trial panels will be installed in three of the original EQC ground improvement trial sites. Both CPT testing and cross-hole geophysical testing will be carried out. It is anticipated that this set of trials will further add to the body of knowledge and confidence around the use of this technology for the improvement of soils beneath existing structures.

8 ACKNOWLEDGEMENTS

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