Stone column ground improvement field trial: A Christchurch case study

D.P. Mahoney & J. Kupec

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ABSTRACT: Foodstuffs South Island Limited is in the process of replacing the Wainoni PAK'n SAVE Supermarket that was damaged beyond economic repair during the Canterbury earthquake sequence. The site is highly susceptible to seismically induced liquefaction and was significantly affected by the strong shaking. The intent is for the rebuild to mitigate the liquefaction risk with stone column ground improvement to provide a stable building area and improve the site performance. A preliminary stone column design was developed using published semi-empirical techniques based on vibro-flotation installation methodology and using first principle soil mechanics. However, due to the residential surroundings of the site and sensitivity around vibration and noise, as well as, the need to keep the existing supermarket operating while constructing the replacement building, a new screw displacement stone column installation methodology was developed with the contractor. This construction method was perceived to generate lower noise levels than conventional ground improvement methods, but critically generated virtually no vibrations. Due to uncertainties associated with a new construction technique and given the site specific sub soil variability both laterally and with depth, a large scale field trial has been completed prior to commencing the main construction sequence. The trial confirmed that; the technique could achieve the required level of ground improvement; confirmed that the new installation technique would consistently work in the highly variable silty sandy subsoil conditions; and, it was used to optimise the stone column spacings and depths. This paper outlines the field trial layout, pre and post-trial proof testing regime, and discusses the trial results and the influence of it on the final construction design. At the time of this paper being written the main construction sequence is well underway.

1 INTRODUCTION

Foodstuffs South Island Limited is in the process of rebuilding the PAK'n SAVE supermarket in Wainoni, Christchurch. The original supermarket building was only seven years old at the time of the earthquakes but has been damaged beyond economic repair during the Canterbury earthquake sequence. The site is located in eastern Christchurch and has suffered significant liquefaction induced ground damage during the major seismic events of 2011. As part of the rebuild process stone column ground improvement has been undertaken at the site in order to mitigate against future liquefaction risk and provide post-earthquake resilience to this important community facility.

The original supermarket building has an approximate footprint area of 5,000m^2 and is founded on shallow strip and pad footings. The building damage was predominately caused by ground movement and differential settlement, which in some places were several hundreds of millimetres. The site is in an urban setting and adjacent to residential houses, a park and a secondary school. In order for the project to remain economically viable the existing supermarket has to stay trading while a replacement store is built in the car park at front of the site.

Due to the project requirements for minimal environmental impact to both the existing supermarket and the neighbouring residential properties a stone column installation method utilising a screw based displacement technique was selected. The chosen installation method is relatively new to the New Zealand market and has limited experience in clean sands. As part of the ground improvement design,

a large scale field trial of various stone column layouts was undertaken in order to confirm the suitability of the proposed ground improvement design from both a technical and environmental perspective. This paper outlines the design of the stone column ground improvement and field trial that was undertaken and how this affected the final ground improvement design.

2 GROUND CONDITIONS

The ground conditions at the site are summarised in Table 1 below.

Table 1. Ground model.

Soil Unit	Depth	Typical Layer Thickness	Material	Typical CPT q_c	Typical Fines Content
1	0–3m	3m	Loose to medium dense fine sand	4-10MPa	0-1%
2	3–5.5m	2.5m	Loose/soft sandy-silt and silt	1-5MPa	15-50%
3	5.5–15m	9.5m	Loose to medium dense medium sand	6-15MPa	<5%
4	15–20m	5m	Dense to very dense sand	15-20MPa	<5%
5	20m–>25m	Over 5m	Medium dense to very dense sand, inter-bedded with firm silt layers	>20MPa	<5%

Groundwater is at a depth of approximately 1.5 to 1.8m below ground level.

During the Ultimate Limit State (ULS) design earthquake scenario, the loose to medium dense sandy and silty-sand layers within Geological Units 1, 2 and 3 have been assessed as being liquefiable to a depth of up to 15 to 16m. Liquefaction induced settlements in the order of 150mm are expected during a ULS design earthquake, and significant surface expression and associated ground damage is expected. The calculated free field settlement generally matches the observed ground damage and reflects the associated building damage.

3 GROUND IMPROVEMENT METHODLGY

In order to provide the future post-earthquake performance that the client requires from the rebuilt facility and considering the geotechnical conditions at the site, ground improvement was selected as the preferred methodology for liquefaction suppression. This method was used successfully in 2010 - 2011 on the Kaiapoi New World rebuild building where the 22 February 2011 earthquake caused only very minor damage to the partially treated ground. To achieve the requested performance level a block of treated and improved ground, using a dense grid of stone columns, was proposed below the footprint of the supermarket building. The treated ground was designed in such a way as to suppress seismically induced liquefaction to a point that a "normal" shallow founded structure can be built on the site.

The site is located within a residential area surrounded by houses, a school and a public park. The primary driver for the client was that the existing PAK'n SAVE supermarket was able to be trade during the rebuild process. Many of the neighbouring buildings, including the existing supermarket, exhibited significant pre-existing earthquake damage. As such the neighbouring sites are highly susceptible vibration nuisance and damage.

Due to the predominantly clean sand subsoils the site is technically well suited to various deep ground

improvement techniques. However, due to the stringent project requirements to minimise and eliminate construction noise, vibration, and general nuisance to the neighbouring sites the ground improvement technique of full displacement stone columns that uses a no vibration and a screw based installation method was selected by the client after a review and field visits to appreciate the impact from other techniques such as vibro floatation or rammed aggregate pier construction. This screw based method of stone column installation is a relatively new technology the New Zealand market.

The installation method works by screwing an outer 500mm diameter mandrel to the founding depth of the stone columns thereby displacing and densifying the surrounding soil. The mandrel is then charged with stone aggregate via a hopper at the top of the rig. The mandrel is then 'backed out' half a turn to expose an internal reverse pitch screw feeder head. The mandrel is then wound out of the ground as the internal screw feeder feeds and compacts gravel into the void created by the mandrel, thereby forming a 600mm diameter stone column.

There is very limited empirical design information for this specific installation methodology. As such, traditional stone column design methodologies were utilised for preliminary design and a full scale field trial was undertaken to confirm the validity of this design. Images of the stone column rig and the screw tool at the bottom of the mandrel are shown in Figure 1 below.

Outer Mandrel

Inner Screw Feeder

Figure 1. Stone Column Installation Rig and Screw Tool

4 STONE COLUMN PRELIMINARY DESIGN

Due to occupancy numbers the replacement building was designed as an Importance Level 3 (IL3) structure in terms of the New Zealand Loadings Code NZS1170.0: 2004. In the absence of any other guidance for IL3 structures the ULS design earthquake has been derived using NZS1170.5: 2004 and the New Zealand Geotechnical Society's *Geotechnical earthquake engineering practice, Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards* (NZGS, 2010). This resulted in a 1,000 year return event design ULS earthquake with a Peak Ground Acceleration (PGA) of 0.44g and a Magnitude (M_w) of 7.5. Although it appears that the seismic shaking is relatively high, nearby seismometers have recorded much higher levels of shaking and hence 0.44g is considered realistic.

The stone columns have been designed to suppress liquefaction through a combination of both ground

densification in the clean sands and densification and stiffening in the inter-bedded silty soils so that under a ULS IL3 design with a calculated factor of safety (FoS) against liquefaction in the order of 1.2. The preliminary stone column sizing and spacing was selected based upon the results of the geotechnical investigation, liquefaction hazard assessment, and using a combination of the stone column design method given by Baez and Martin (1993) and first principles of weight-volume and mass-volume relationships.

The stone columns are being installed to a nominal depth of 10.5m to create a sufficiently thick crust to remove the shallow liquefaction hazard below the building and suppress the surface expression of any deep liquefaction. Thereby reducing the expected liquefaction induced ground movements to acceptable levels, i.e. in the order of tens of millimetres as opposed to the non-improved ground where hundreds of millimetres of settlement would be expected and was observed on site.

The ability for the stone columns to reduce liquefaction susceptibility through drainage was specifically not taken into account in design as the drainage properties of the columns are heavily affected by the installation process, and it is unknown for how many liquefaction cycles the columns could act as drains for before clogging with silty-sandy ejecta material. Clogging of the outer row of stone columns was observed on the Kaiapoi New World supermarket rebuild where after one large earthquake the stone column was effectively choked with fines. Although this is unlikely to affect the load bearing ability, hydraulic conductivity would be severely compromised. Consideration to sleeving stone columns in geofabric was given but rejected due to the chosen installation method.

Due to the absence of any dense non-liquefiable layer, load bearing layer, in the upper 15m of the soil profile the columns do not have the capacity to act as load transfer columns. As such all the liquefaction mitigation must be achieved with ground improvement, through densification and stiffening only.

5 FIELD TRIAL

Due to the size of the ground improvement area, over 9,000m^2, and the fact that the preliminary design is based upon a semi-empirical design methodology for vibro replacement installation techniques, a large scale field trial was undertaken to confirm the required levels of ground improvement to achieve adequate liquefaction suppression; to confirm the suitability of the screw based stone column installation technique; and to optimise stone column spacing and layout to determine project cost and timeframes.

5.1 Layout

The field trial was comprised of the installation of three trial grids. Each trial grid was comprised of various spacing, layout and depth combinations with 23 to 59 stone columns at each trial grid. Each trial grid had a footprint of approximately 7m by 9m.

The locations of the trial grids were selected due to:

- The subsoil stratigraphy being representative of the critical design case i.e. contained an approximately 2m thick silty-sandy layer between 3 to 5m below ground level.

- It being in an area that was not directly below the footprint of the proposed new supermarket building in case adequate ground improvement was not achieved.

- It being in a dead end of the car park in order to minimise disruption to on-going supermarket operations.

The critical design consideration for each of the trial layouts was:

- Layout 1 – 600mm diameter stone columns on 1.65m triangular grid spacing. Designed to theoretically achieve densification of the silty layer (soil Unit 2) with a design replacement ratio of 12%.

- Layout 2 – A variable 700mm/600mm diameter stone columns on a 1.9m triangular grid

spacing. Designed to theoretically achieve densification of both the silty layer (soil Unit 2) and the lower sand material (soil Unit 3) with a design replacement ratio of 12% in the upper 5.5m and 8% below 5.5m depth.

- Layout 3 – Variable length 600mm diameter stone columns, 10.5m deep columns on a 2m triangular grid spacing with infill 5.5m deep columns on a 1.1m triangular grid spacing. Designed to theoretically achieve densification of both the silty layer (soil Unit 2) and the lower sand material (soil Unit 3) with a design replacement ratio of 27% in the upper 5.5m and 8% below 5.5m depth.

Schematics of the three grid layouts are presented in Figures 2 to 4 below.
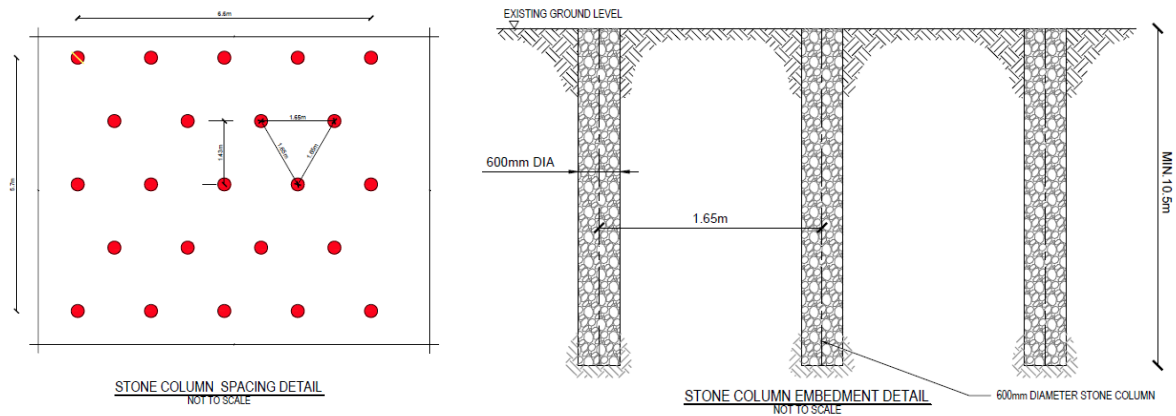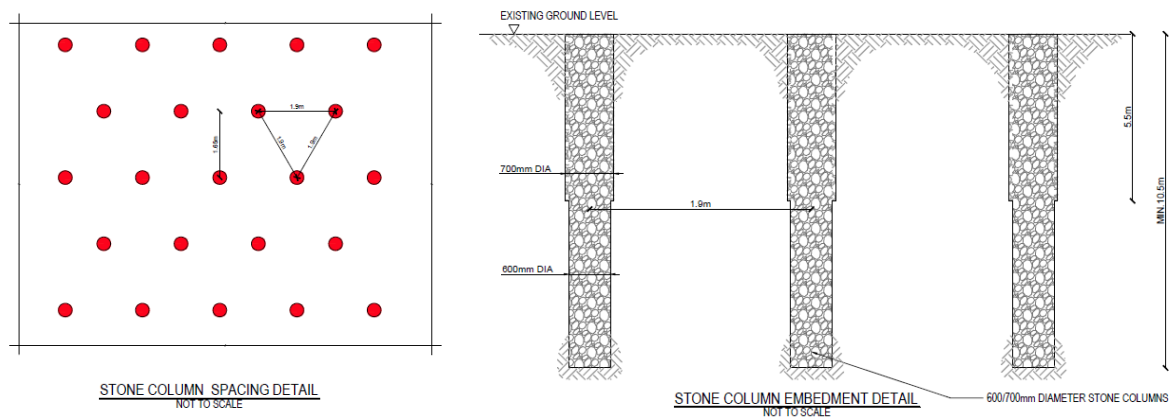

Figure 2. Stone Column Trial Layout 1

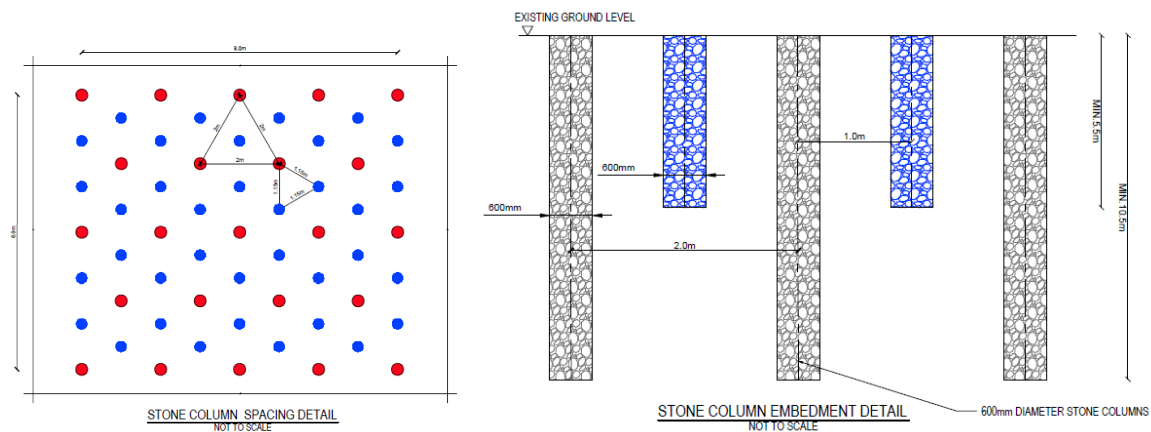**Figure 3. Stone Column Trial Layout 2**

Figure 4. Stone Column Trial Layout 3

Alternative layouts were considered such as variable spacing based upon area specific ground conditions. However, the selected grids and spacings were preferable due to the repeatability of the column installation and several areas of similar densification due to consistent column spacing.

5.2 Testing

A series of geotechnical tests were undertaken to confirm the ground conditions pre-trial and assess the degree of improvement achieved post-trial for each of the three stone column layout options, and also investigate how the improved ground potential changed with time, due to pore water pressure dissipation and installation of neighbouring column grids.

Pre-trial these tests comprised:

- A borehole with Standard Penetrometer Testing (SPT) at 1.5m centres to 12m depth at the centre of each trial grid.

- A Piezocone Penetrometer Tests (CPTu) undertaken at a distance half way between the central column and the column directly adjacent.

Post-trial these tests comprised:

- A borehole with SPT testing at 0.75m centres undertaken down the centre of the centre column in each trial grid.

- At one week intervals post installation CPTu soundings were undertaken around the central column in each trial grid. The CPTu soundings were undertaken at a distance half way between the central column and the column directly adjacent.

A typical post–trial CPTu profile for Trial Area 1 is presented in Figure 5 below.
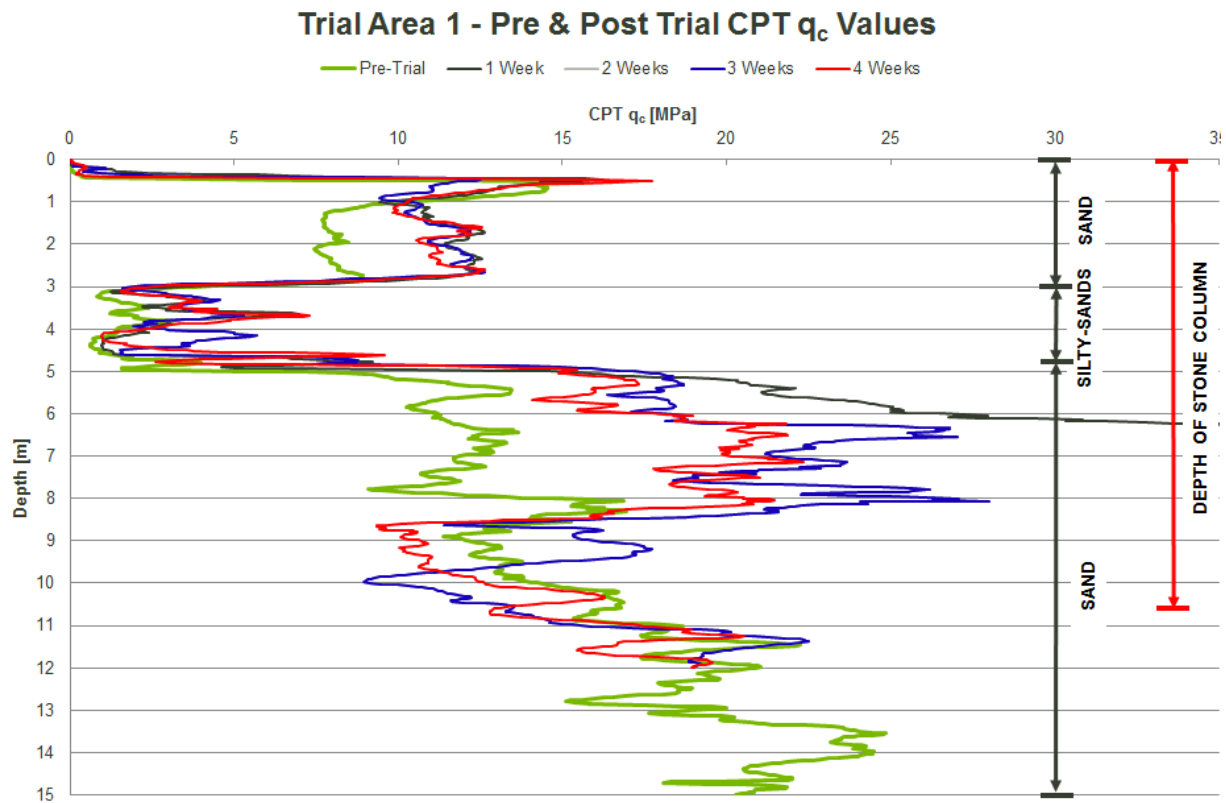
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Figure 5. Pre and Post-Trial CPT q_c Plots Trial Layout 1

The following general trend was observed with the pre and post-trial CPTu testing as outlined in Table 2 below:

Table 2. Ground Improvement Summary

Depth	Material	Pre-Trial CPTu q_c	Post-Trial CPTu q_c
0–3m	Sand	7MPa	12-18MPa
3–5m	Silty-Sand and Silt	2-5MPa	2-5MPa
5–9m	Sand	9-12MPa	18->30MPa
9-10.5m	Sand	10-12MPa	12-15MPa

6 REVISED STONE COLUMN SPACING

The initial stone column spacing was determined based upon the method outlined by Baez and Martin (1993). The design relied primarily upon ground densification as the mechanism for ground improvement. The use of ground stiffening and drainage effects were not considered in this initial design.

Liquefaction hazard assessment was run on the post-installation CPT results using the same method and design earthquake scenario as the initial design. Based upon the results of liquefaction assessment under the ULS design earthquake scenario the following key points were identified:

- The upper sand layer was sufficiently densified by all three layout options to suppress liquefaction.

- The silty layer from typically 3 to 5m depth was not densified sufficiently by trial layout options 1 and 2 to suppress liquefaction and softening.

- The lower sand layer form 5 to 9m depth was sufficiently densified by all three layout options to suppress liquefaction.

- Limited if any soil densification was achieved by the bottom 1m or so of the stone columns.

As such the theoretically liquefiable silty layer from 3 to 5m depth governed the ground improvement design. When considering total length of stone column installed, construction costs and programme with the client and contractor it was decided that a single length 600mm diameter stone column was going to be the most practicable construction solution. As such, the stiffening capacity of the stone columns in the silt layer was utilised to help suppress liquefaction, maximise stone column spacing and hence minimise the number of stone columns required.

Ground stiffness parameters were derived from the silty-sandy material using the measured q_c values from the post-trial CPTu testing and the published correlations (Bowles, 1996). The column stiffness has been developed using the measured SPT 'N' values from the post-installation borehole tests undertaken down the stone columns in the centre of each trail area, and published stiffness correlations (Bowles, 1996, and Kramer, 1996). An optimised 'production' stone column triangular spacing 1.85m was derived using a combination of the calculated stiffness parameters (for the treated silty soil, and stone column); the results of the post-trial liquefaction assessments; and, the method of Baez and Martin (1993) for calculating liquefaction mitigation through ground stiffening. This corresponds to a replacement ratio of 10%. Again the drainage effects of the stone columns are not being relied upon for liquefaction mitigation. However, despite discarding the mitigating effects of drainage, it is acknowledged that the stone columns could act as preferential drainage paths during a major earthquake. As such, the foundation design accounts for ground water ejecta coming up the stone columns through the construction of a 0.5m thick drainage blanket that will be located at the top of the stone columns below the building footprint.

7 CONCLUSIONS

The preliminary stone column sizes and spacing were selected based upon the results of the geotechnical investigation, liquefaction hazard assessment, and using a combination of a semi-empirical stone column design methodology based upon vibro-replacement techniques and first principles of weight-volume and mass-volume relationships.

Due to project environmental constraints a screw based, non-vibratory, stone column installation technique was employed. As such a series of field trials were undertaken with different stone column diameter and layout combinations. This trial was undertaken to confirm the required levels of ground improvement to suppress liquefaction at ULS levels of seismic shaking; the suitability of the screw based stone column installation technique; and optimise the stone column grid layout to inform on project cost and programme.

Upon completing the stone column field trial the following key points were identified:

- Significant ground densification was achieved in the clean sandy soils.

- As anticipated, based upon the semi-empirical design method, densification of the silty soils was very limited irrespective of the replacement ratio in these soils. As such, stiffening was utilised for ground improvement in this material.

- The trial allowed confirmation of the level of ground improvement that could be achieved and from this optimisation of the ground improvement requirements.

- The finalised construction design was 600mm diameter stone columns on a 1.85m triangular spacing with a replacement ration of 10%.

At the time of writing, construction has commenced at site and approximately 20% of the production

stone columns have been installed. During production the design level of ground improvement achieved during the trial is consistently being achieved during production. However, production rates are significantly less than initially anticipated. This is attributed to the variability in density of the lower sand (Unit 3) layer across the site. The effect that Unit 3 has had on stone column production rates was not identified in the field trial process.

8 ACKNOWLEDGEMENTS

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