Performance of Rammed Aggregate Piers as a soil densification method in sandy and silty soils: experience from the Christchurch rebuild

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ABSTRACT

Following the Canterbury Earthquake Sequence (2010-2011), widespread areas within Christchurch, New Zealand were affected by liquefaction. Soil densification using Rammed Aggregate Piers® (RAP) elements were widely used as a liquefaction mitigation measure on numerous new residential and commercial developments. Cone Penetration Test (CPT) is commonly used in New Zealand to assess the degree of densification following the installation of all compacted gravel column technologies. In this paper, a large database of CPT data spread among 80 sites across Christchurch was analyzed, covering a wide range of ground conditions ranging from clean sandy materials to fine grained soils. Post RAP installation CPT's are compared to pre RAP installation CPT's to estimate the degree of ground improvement. Changes in cone resistance q_c following RAP installation were assessed. Correlations with different factors such as soil type, depth, consistency/density and time were also investigated in an attempt to predict the degree of improvement.

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1 INTRODUCTION

1.1 Prediction of the Improvement for Performance Based Design

This study is based on the comparison of CPT data undertaken before and after installation of highly compacted aggregate columns in different soil conditions across Christchurch, New Zealand.

The aim of this paper is to help designers predicting the degree of improvement measured in terms of cone tip resistance increase and predicting the expected degree of densification to mitigate earthquake-induced liquefaction.

In this paper, cone tip resistance change for different soil types is assessed for the design stage prediction of the anticipate ground performance against liquefaction during a future seismic event. This study has been undertaken within the Christchurch rebuild context following the 2010 and 2011 devastating earthquakes that caused major expression of liquefaction in some areas. This study is the start of a series of work to investigate the efficiency and determine the design input parameters, and its influence, for the proposed ground treatment method, for the Christchurch alluvial and marine deposits.

1.2 Scene Settings

Christchurch, New Zealand, was affected by a series of earthquakes between 2010 and 2011 known as the Canterbury Earthquake Sequence (CES). The most damaging earthquake occurred on 22 February 2011 with the highest recorded PGA of 2.2g (vertical component) and around 1g horizontally in the CBD. Christchurch's central city and eastern suburbs were significantly affected by liquefaction-induced damage to buildings and infrastructure. The liquefaction caused ground movement, damaging many foundations and destroying infrastructure. A large volume of silt and sand was ejected on to the surface. Approximately 550,000 tonnes of liquefaction ejecta was removed from the greater Christchurch area between September 2010 and August 2011 (Villemure et al. 2012).

1.3 Geological Soil Conditions

Christchurch is underlain by a series of interbedded coarse and fine grained sedimentary deposits (Forsyth et al. 2008). The gravels and sands have been laid down by rivers, coastal and wind-related processes. Fine grained sediments have been deposited in near offshore marine environments, estuaries, lakes, wetlands and on flood plains. Organic deposits including peat, wood and shells are distributed throughout the deposits underlying the Christchurch area.

Most of sites treated in this study had typical ground conditions as follows:

- Interbedded layers of sand and silt, mainly located in the center and the western side of Christchurch and associated with alluvial deposits.
- Relatively clean sand sites mainly located on the eastern side of Christchurch and associated with beach deposits.
- Interbedded layers of sand and silt deposits overlain by soft organic silt, mainly located in former swamp areas.

1.4 Christchurch Rebuild Post-earthquake land classification

Among the 180,000 homes in greater Christchurch, up to 8000 properties were mapped as "Red Zone" (red as shown in Figure 1) meaning rebuilding on them is unlikely to take place for a prolonged period. 6000 of those properties were damaged due to liquefaction.
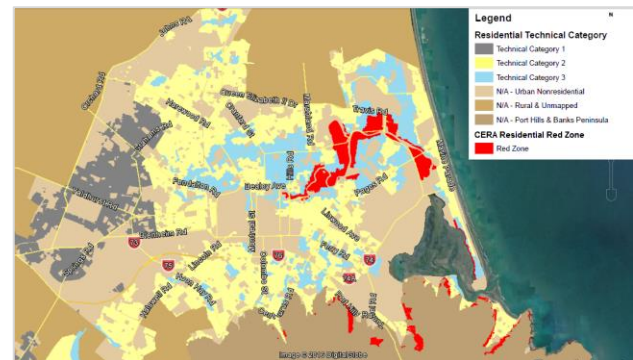

Figure 1. Map of greater Christchurch area showing red and TC zones developed for residential properties (MBIE, 2015)

For the remaining properties, the government has classified the residential land into three technical categories (TC):

- unlikely future damage to the land due to liquefaction (TC1, grey shown in Figure 1);
- liquefaction damage is possible in future significant earthquakes (TC2, yellow shown in Figure 1);
- liquefaction is possible in future large earthquake (TC3, blue shown in Figure 1)

Most of the Christchurch Building District (CBD) was classified as "Urban Non Residential" to emphasize the need for a site specific engineering input and design for foundation assessments, building retrofit and new foundation designs.

2 REBUILD USING RAP GROUND IMPROVEMENT

2.1 Soil Densification as a Countermeasure for Liquefaction

Following the CES, the Ministry of Business, Innovation and Employment (MBIE) proposed several alternatives as proposed for the repair or rebuilding of house foundations within greater Christchurch to address the potential for future damage from liquefaction in land classified as TC2 and TC3 (MBIE, 2015). A full-scale series of instrumented shaking trial (EQC, 2015) demonstrated that columns of highly compacted aggregate or Rammed Aggregate

Piers® (RAP) is an effective method for remediating land susceptible to liquefaction. The trial showed that the RAP displacement method effectively densified clean sand deposits with Ic < 1.8 but did not provide measurable densification for the upper soil horizon with Ic > 1.8 (Wissmann et al. 2015) within the duration of the trial. The RAPs were shown by the shaking tests and measurement of the insitu cyclic shear strains to provide a composite stiffening effect which slowed the development of elevated pore-water pressures in soils with I_c > 1.8.

Since the trials in 2013 (EQC, 2015), more than 80 sites have been treated for liquefaction using RAPs as shown on Figure 2.
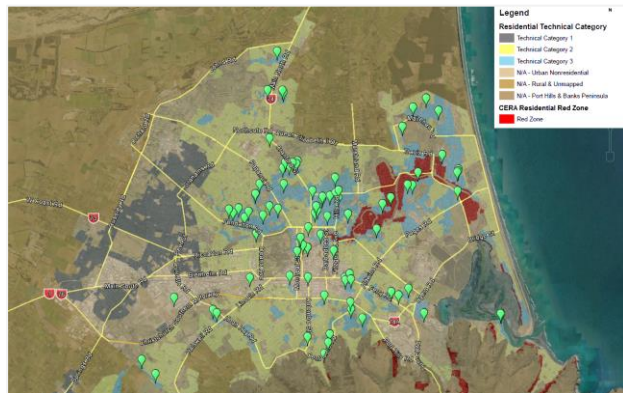

Figure 2. Map of sites with ground improvement using RAP

2.2   Rammed Aggregate Piers®

RAP elements are constructed using displacement techniques with an excavator-mounted mobilram base machine fitted with a high frequency (30 to 40 Hz) vibratory hammer. The base machine drives a 250 to 300 mm outside diameter open-ended pipe mandrel fitted with a unique specially-designed 350 to 400 mm diameter tamper foot into the ground. The method uses hydraulic crowd pressure and vertical vibratory hammer energy to displace and densify the liquefiable soils. Crushed gravel (typically graded at 12 to 40 mm in particle size) is fed through the mandrel from a top mounted hopper and compacted in the displaced cavities to create approximately 600 mm diameter, dense, stiff, aggregate pier elements (Figure 3).

RAPs are typically designed and built using either a triangular or square layout pattern (Figure 4), adjusting the center to center spacing to achieve the target area replacement ratio. The area replacement ratio (ARR) is given by the following equation (The Japanese Geotechnical Society, 1998):

- $ARR = \dfrac{Ac}{S^2}$ for a square configuration   [1]

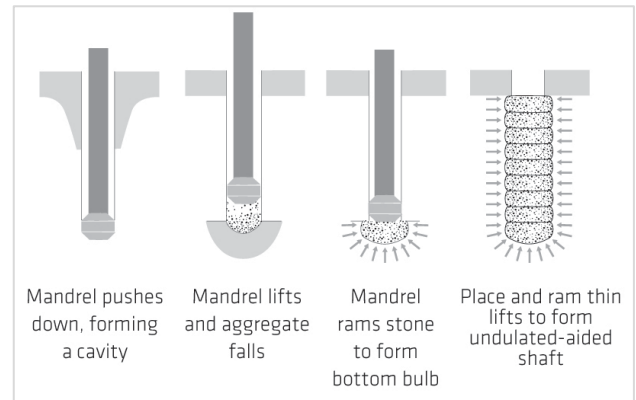- $ARR = \dfrac{Ac*2}{S^2*\sqrt{3}}$ for a triangular configuration [2]

Figure 3. RAP ground improvement construction method (Wissmann et al. 2015).

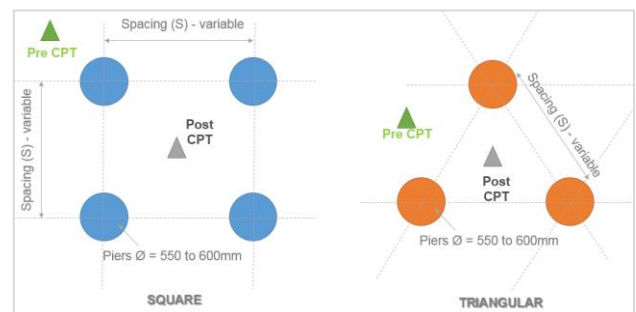With Ac the area of the pier dependent on the diameter and S the spacing between piers centre to centre.

Figure 4. Cell of RAP comprising four piers in a square pattern and three piers in a triangular pattern. Location of typical pre and post (verification) CPT.

ARR between 6% and 14% were built with the majority of the sites having an ARR of 8% corresponding to the minimum ARR recommended by MBIE (2015) supported by post improvement CPT testing. The CPT is used as the preferred method for measuring the effectiveness of in situ densification due to its widespread usage across Christchurch and its generally accepted basis for simplified liquefaction triggering analysis methodologies.

2.3   Construction sequencing

As part of the initial geotechnical damage assessment, pre-treatment Cone Penetration Tests (CPT) are always undertaken prior to RAP construction for design and performance objectives.

Due to constraints associated with the construction planning of the new building, verification ("post") CPTs are typically undertaken within two weeks after construction and usually located in the middle of a cell as shown on Figure 4 (i.e., at the location of least improvement). In some cases, verification CPTs were undertaken one

month after treatment but this was rare and usually due to an unexpected delay in subsequent construction.

3 PERFORMANCE ASSESSMENT METHODOLOGY

3.1 Cone penetration resistance

Cone penetration test raw data including the tip resistance, the sleeve friction and the pore pressure measurement are usually the only data provided to the engineer at design stage. Soil densification is the main objective of ground improvement design and the performance assessment presented in this paper is based primarily on the measured cone tip resistance q_c which relates to the relative density (Jamiolkowski et al. 1988). Although the corrected tip cone resistance q_t can be more relevant especially for clayey soils, the study has been conducted using q_c for the following reasons:

- q_c values, within the range of soils expected to respond to densification (sands and silty sands with water pressure close to the hydrostatic pressure) will not change significantly if corrected for pore water pressure.
- The groundwater regime will potentially change following installation of RAP and the use of the pore water pressure measurement independently of q_c maybe a potential indicator of that effect. This aspect is currently under development within Golder.

3.2 Selection criteria

The performance of RAPs as a densification method was assessed by comparing pre and post CPT traces. As discussed previously, because of the nature of the deposition processes, Christchurch soils exhibit a large spatial variability often at the scale of the site. Among the 80 sites where RAP was used for liquefaction mitigation in Christchurch since 2011, the quality of the pre and post CPT data was checked for the following criteria:

- Quality of the CPT data (pre and post), i.e., data containing obvious errors were disregarded.
- Distance between pre and post CPT (maximum distance of about 10 m).
- Consistency of soil layers between pre and post CPT.

As shown in Figure 5, and for each post verification CPT, a careful correlation of soil layers was undertaken with the pre CPTs available for the site. As the geotechnical properties of the interbedded deposits in Christchurch vary laterally, discrete layers were selected from CPT data used in our analyses. It should be noted that the first meter of the ground profile below the ground surface was systematically excluded to eliminate any potential bias from the initial penetration phase whereby q_c increases with penetration depth.

In total, 81 verification CPTs from 20 sites were used in this assessment from which a total number of 244 discrete soil layers were identified.

3.3 Soil behavior type differentiation

The normalized soil behavior type index as defined by Robertson and Wride (1998) was used in this study as a differentiator. Wissmann et al. (2015) suggested that soils with a soil behavior type index (I_c) lower than 1.8 exhibited greater densification that those with an I_c greater than 1.8. The data comparison was therefore split into two categories based on their soil behavior type index:

- I_c < 1.8 corresponding to relatively clean sand.
- I_c > 1.8 corresponding to silty layers with some or minor sands corresponding in most of cases to soft to firm silt of young alluvial deposits.

Since I_c > 2.6 is considered as the upper bound cut-off value for liquefiable soils (Robertson and Wride 1998) and most of residential Christchurch rebuild is based on MBIE (2015) stating that soil with I_c > 2.6 are generally regarded as non-liquefiable, no analysis on this type of soil was undertaken for this study.

3.4 Bulb effect and effective treatment depth

The construction sequence of a pier involves the formation of a bulb at the base of the pier which acts as a stabilized layer for the compaction of subsequent aggregate lifts (Figure 5). Consequently, densification is regularly observed at depths exceeding the target treatment depth. A distinction was therefore made between layers located entirely along the pier (Type A), layers extending below the design treatment depth (Type B) and layers located entirely below the design treatment depth (Type C).

Type B layers extending approximately up to 2 piers diameters below treatment depth and Type C layers extending up to 2 to 7 piers diameter below treatment depth could be observed. This suggests a consistent level of improvement may be expected several diameters below the tip of the columns with a zone of influence extending further. . This phenomena is currently under study and will be developed in the near future.

The typical treatment depth for sites included in our analyses was between 4 m and 9 m below ground level (bgl) with a ground water table for liquefaction assessment generally between 0.8 and 1.2m bgl.

4 PERFORMANCE OF RAP GROUND IMPROVEMENT

For each selected layer, an improvement ratio (Qc), was introduced as an indicator of improvement performance as the ratio between the post improvement q_c averaged over the layer thickness and the pre improvement average:

$$Qc = \frac{q_c\,post}{q_c\,pre} \qquad [1]$$

Influence of various parameters such as soil behavior type, relative density or overburden effective stress on the achieved performance ratio are discussed in the following sections.

4.1 Soil behavior type index (I_c)

As discussed previously, the dataset was split between clean sandy soils (I_c <1.8) and siltier soil materials (I_c >1.8). Figure 6 shows achieved performance as a function of pre improvement tip resistance q_c for the two soil types. Data population shown on Figure 6 represents 23 individual sites, 50 pre and 80 post CPTs to depths of between 4 to 9m bgl, and approximately 163 different layers with 68 layers for I_c<1.8 and 95 layers for 1.8<I_c<2.6. The plot denotes that Qc ratio depending on the pre q_c for I_c<1.8 is comprised between 1.2 and 2.1 while the distribution for I_c >1.8 is much wider.

For I_c<1.8, as pre improvement tip resistance increases, an overall decrease of improvement performance is observed which could be attributed to the increasing effort required to densify soils as the initial density increases. Results are further discussed in section 4.4.

For I_c >1.8, the data in Figure 6 shows a greater scatter in the Qc ratio (up to 3.5 or greater) down to less than 1 for soft silty soils which are further discussed in section 4.5.
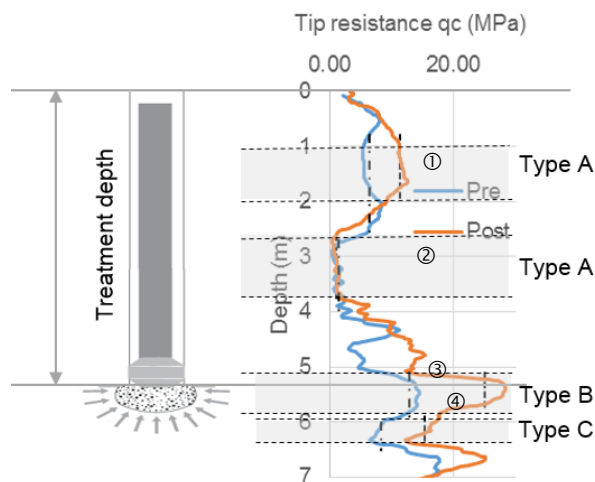

Figure 6. Qc ratio as a function of pre q_c

Figure 5. Example of correlation for pre and post CPT traces and definition of three layer types.

4.2 Square vs. triangular layout pattern

A comparison of Qc ratio as a function of the cell configuration (Figure 4) indicates that generally the same level of improvement is observed for square or triangular pattern (Figure 7).

4.3 Area replacement ratio

Most sites constructed to date and used in this assessment were based on an area replacement ratio of 8% as per MBIE Guidance recommended procedure (MBIE, 2015). Designing using a performance-based approach, other replacement ratios were used, ranging from 6% to 14%. However, the number of data collected to date is not sufficient to derive a meaningful trend to relate performance ratio to the area replacement ratio.
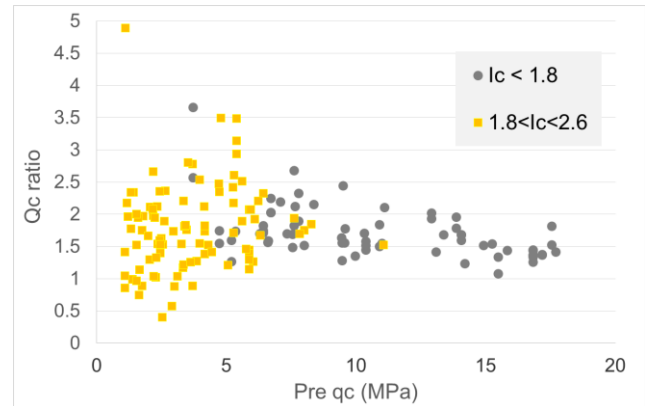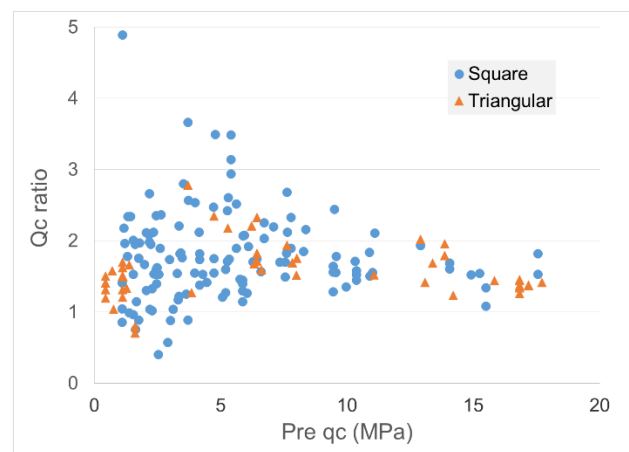

Figure 7. Comparison of achieved improvement performance for square (132 data points) and triangular (31 data points) layout configurations

Figure 8 shows the improvement ratio distinguishing sites with an ARR = 8%, sites with an ARR < 8% and sites with an ARR > 8%. The distribution of Qc ratio for ARR = 8% is larger than Qc ratio for ARR>8% and ARR<8% but this set of data has been collected over 19 sites while data for ARR<8% and ARR>8% represents 2 and 2 different sites respectively.

Generally, similar trends are observed for all three classes with higher replacement ratios showing generally greater improvement although additional data would be required to clarify this trend.

4.4 Improvement performance of RAPs on clean sand (I_c <1.8)

4.4.1 Influence of relative density

Post improvement Qc ratio for clean sands (I_c < 1.8 and 8% ARR) is shown as a function or relative density on Figure 9. The first observation is the absence of loose or very loose sand. Although these types of soils are present in Christchurch and have been encountered in some sites as part of this study, the I_c value indicates that those soils

tend to be categorized as silty materials. Also loose sand tends to be present at shallow depths and for the reason explained in section 3, layers located within the upper meter of ground profile have not been selected for this study.
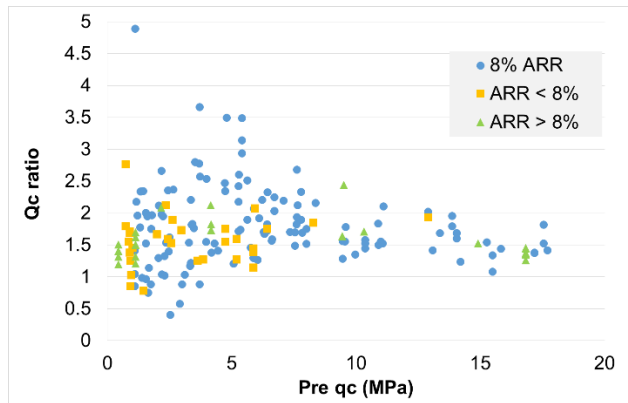

Figure 8. Qc ratio as a function of pre q_c for all ARR - 26 layers for ARR<8% (i.e. 16% of the full dataset), 124 layers for ARR of 8% (i.e. 76% of the full dataset) and 13 layers for ARR>8% (i.e. 8% of the full dataset).

The general shape indicates a Qc ratio decreasing with the relative density of the soil prior to pier (i.e. RAP) installation. It should be noted that some outlier points with Qc ratio greater than 2.5 fall outside the overall pattern of the scatter plot and were obtained on the same site. Scatter plots for data points within Type B and Type C layers indicate that Qc is similar for both categories but they are located in the lower range of the improvement (Figure 9). Those points reflect the influence of the bulb in the underlying layers.
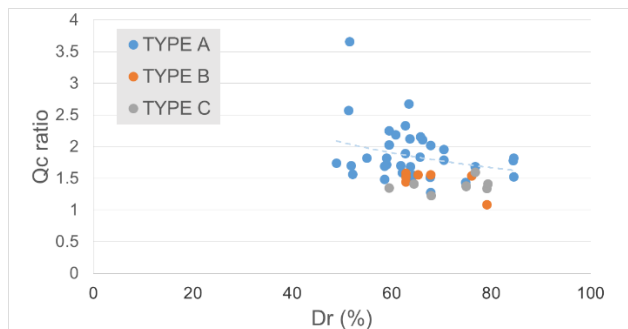

Figure 9. Qc ratio as a function of relative density D_r for I_c<1.8

4.4.2   Measured improvement function of confinement

The effectiveness of ground improvement was also examined as a function of confinement. The vertical effective stress rather than the horizontal effective stress was considered as an indicator of the confinement.
The trend in Qc ratio with increasing overburden effective stress is shown in Figure 10 and indicates two distinct correlations for σ'$_{vo}$ <30 kPa and for σ'$_{vo}$ > 30 kPa.

Two average linear trends have been extrapolated and are given by equations 3 and 4 as follows:

For σ'_{vo} <30 kPa :

$$Q_c\ ratio = 4.0 - 0.076 * σ'vo \qquad [3]$$

With maximum top of layer considered for this equation in the order of 1m bgl.

For σ'_{vo} >30 kPa :

$$Q_c\ ratio = 1.69 \qquad [4]$$

In addition, 80% confidence level in Qc ratio was calculated for both trends and can be derived from equations [3] and [4] applying a 0.14 vertical offset (Figure 10).
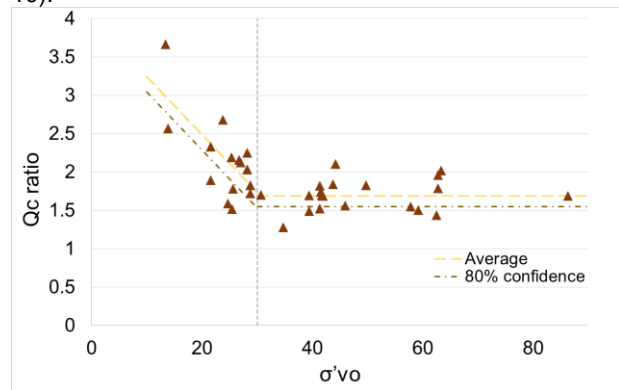

Figure 10. Qc ratio function of σ'_{vo} for I_c<1.8

4.5   Improvement performance of RAPs in silty soils (1.8< I_c<2.6)

4.5.1   Variation of Qc ratio for silty soils

For silty soils, with I_c between 1.8 and 2.4, the 95 data points in Figure 6 exhibit a larger scatter than clean sand. The Qc ratio was found to vary between 0.4 and 4.8 with 31% greater than 2 and 9% lower than 1 (post column installation q_c lower than the pre installation q_c). As shown on Figure 11, Qc ratio for silty soil generally increases with q_c pre and reach a peak at around 6 MPa which coincides with the transition to clean sand (I_c<1.8). Qc ratio tends to decrease for clean sand with increase of q_c pre. This observation could raise comments about the role of the fine matrix with regards to densification and suggests the importance of undertaking sieve analysis in laboratory to assess the gradation of those soils. Unfortunately, no laboratory tests have been undertaken to correlate with the data used in this study.
Over the 95 data plotted on Figures 6, 7 and 8, nine values for Qc ratio are less than 1.0 ranging between 0.4 to 0.98. Those data are representative of two sites with layers located at shallow depth around 1.5m bgl. No change in the construction methodology was noted for

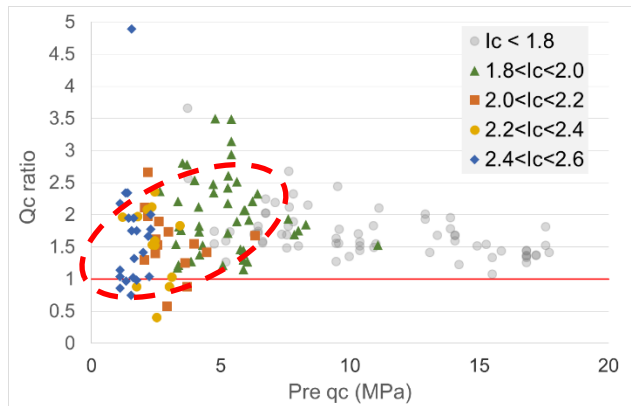those sites and q_c increase was observed at greater depths.

Figure 11. Qc ratio function of pre q_c for silty soils

4.5.2 Time-dependent measured improvement

For most of sites where RAP was used as liquefaction mitigation in silty soils, the level of improvement is often not sufficient to fully prevent liquefaction triggering using the Boulanger and Idriss (2014) liquefaction triggering assessment methodology.

The time component between improvement and testing is an important factor in the results obtained of measured densification/consolidation in silty soils.

Figure 13 shows the increase of the Qc ratio over time for four different sites. The ratio increases after a few weeks and the trend indicates that the dissipation of excess of pore water pressure is unlikely to be complete.

Qc ratio less than 1.0 observed in Figure 11 and Figure 12 (Site 4) could potentially be due to the pore water pressure increase under the effect of vibration induced by the ramming equipment that reduces the effective stresses. Qc ratio increase at Site 4 at two weeks post improvement (Figure 12) reach almost 1.0. This suggest that the pore water pressure has dissipated partially.

Additional investigation, where practical, should be undertaken few weeks after installation to confirm the effect of the vibration on the pore water pressure of the improved soils.

4.5.3 Measured improvement as a function of confinement

Figure 13 indicates a large scatter of data points for Qc ratio as a function of σ'_{vo} for silty soils. The scatter plot for soils with $1.8 < I_c < 2.4$ indicates a negative weak correlation with a Qc ratio generally decreasing with increase of I_c.

No correlation is observed for scatter with I_c greater than 2.4.

The linear trends considering ARR of 8% for soil with Ic comprised between 1.8 and 2.4 have been extrapolated and are given by equations 5, 6 and 7 as follows:

For $1.8 < I_c < 2.0$:

$$Q_c\ ratio = 2.7 - 0.015 * \sigma'vo\ \text{(kPa)}$$
[5]

$$With\ R^2 = 0.16$$

For $2.0 < I_c < 2.2$

$$Q_c\ ratio = 2.0 - 0.01 * \sigma'vo\ \text{(kPa)}$$
[6]

$$With\ R^2 = 0.06$$

For $2.2 < I_c < 2.4$

$$Q_c\ ratio = 1.7 - 0.007 * \sigma'vo\ \text{(kPa)}$$
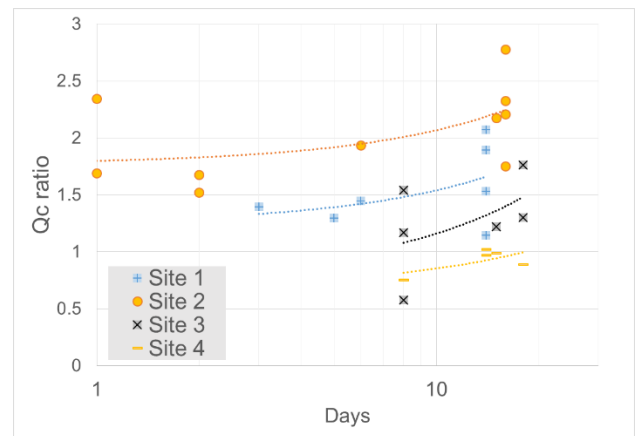[7]

$$With\ R^2 = 0.03$$

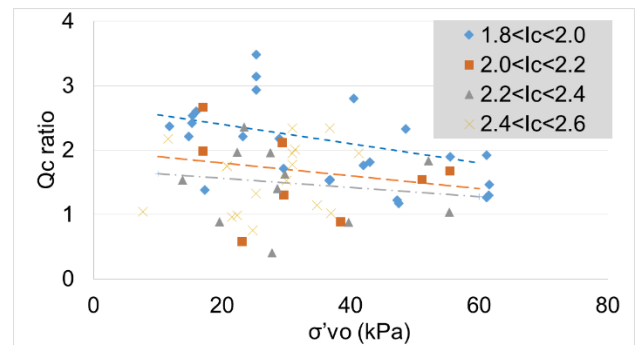Figure 12. Qc ratio increased vs days post treatment

Figure 13. Qc ratio as a function of σ'_{vo} for I_c $1.8 < I_c < 2.6$

Equations 5, 6 and 7 reflect the decreasing and relatively minor influence on Qc ratio of the effective overburden stress as I_c increases. It is likely that these results may be biased due to insufficient consolidation time and may therefore be influenced by other parameters such as the fines content, drainage conditions, and thickness of the treated layer as addressed earlier. Considering the scatter shown on Figure 13, equations 5, 6 and 7 should not be used for design purposes in a quantitative way.

5   DISCUSSION AND CONCLUSIONS

The objective of this study is to predict the degree of densification improvement using Rammed Aggregate Piers as a function of the area replacement ratio and of soil behavior type index (i.e. $I_c<1.8$ and $I_c>1.8$).

Overall, this study confirms the previous study undertaken by Wissmann et al. (2015) within the context of the EQC trial (2015) and shows that densification can be reliably quantify with CPT q_c measurement in clean sand ($I_c<1.8$). It was observed that the amount of densification resulting from column installation varies with relative density and overburden pressure. Relationships are proposed to predict the expected densification.

In the case of silty soils, it also demonstrates that in most cases, post RAP improvement CPT q_c values increase in silty soils indicating that densification has occurred. Yet, some of the data analysis has shown that q_c has decreased post ground improvement. This highlight the importance to accurately monitor the construction work at the site and note any significant changes in the construction process or particular observation of the ground during construction. It also argue that method of installation can alter the ground water regime that could lead to a misinterpretation of the achieved improvement. Further study is on-going to study the effect of the pore water pressure on the measured densification.

At this stage, most of improved sites were designed using an 8% area replacement ratio (ARR). While some sites have been designed using lower and higher values of ARR, the number of data is not high enough to extrapolate other trends. One of the main observations is that the Qc ratio for ARR greater and lower than 8% are not outliers in comparison with the scatter plot for 8% ARR and as such suggest that slightly less to similar level of improvement as for 8% could be expected.

By comparing the Qc ratio as a function of different parameters, the increase of post treatment q_c as a function of the effective overburden stress (σ'_{vo}) appears to be the strongest correlation to be used for design purposes in clean sand. The scatter plot for silty soils as a function of effective overburden stress indicates a weaker correlation between a reducing Qc ratio and depth occurs compared with clean sands ($I_c < 1.8$). Silty soils tend to adopt a more constant Qc ratio instead of a negative Qc ratio correlation as observed for sandy soils particularly for σ'_{vo} less than 30kPa.

Further research could be undertaken to consider the normalized CPT tip resistance q_{c1n} proposed by Boulanger (2003) which automatically eliminates the influence of the soil's effective overburden pressure.

Another important observation from the study is the influence on the measured Qc ratio in silty soils of the time component or delay between treatment and the post CPT work. The CPT tests were usually performed within two weeks after improvement and a clear positive correlation of Qc ratio can be seen over time. While it is difficult to extend the time between ground improvement works and testing in a practical way (due to client desires to commence building), this study shows the benefit of allowing for a greater period of rest post treatment for silty soils.

The correlations given in this paper to estimate post q_c using RAP are extrapolated from the worst case scenario as data collected were measured in the middle of treatment cells and greater densification can be expected closer to the columns. These correlations can be used as an assessment tool to help predicting liquefaction mitigation that can be expected using RAPs during ground improvement design.

6   ACKNOWLEDGMENTS

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