

Rammed Aggregate Pier Ground Improvement as a Liquefaction Mitigation Method in Sandy and Silty Soils

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ABSTRACT

Ground improvement methods have been used for over 70 years to densify loose sands prone to liquefaction. Although these methods reduce liquefaction triggering potential and settlement in densifiable soil, such as loose clean sand, their impacts on soils that are difficult to densify, such as silty soils, are not well understood. This paper examines the results of full scale testing performed for Rammed Aggregate Pier™ treated soil in Christchurch, New Zealand carried out as part of a large scale study by the New Zealand Earthquake Commission. The paper describes pier construction, and outlines test results including pre- and post-installation cone penetration test tip resistances, crosshole shear wave velocity, and vibroseis shaking tests. The results indicate that soil densification may be considered to be the primary liquefaction mitigation mechanism in soils with a soil behavior type index, $I_c < 1.8$, and that composite dynamic stiffness of the RAP-treated soil likely dominates the liquefaction resistance mechanism in soils with $I_c > 1.8$. This paper is of particular significance because it provides a well-documented link between a widely used ground improvement method and the mechanisms involved in liquefaction mitigation.

Introduction

Christchurch, New Zealand is founded on a complex, interlayered sequence of alluvial soils vulnerable to liquefaction-induced land damage from moderate to severe earthquake events. Widespread ground surface deformation from liquefaction-induced differential and total settlement and lateral spreading occurred during the 2010 to 2011 Canterbury Earthquake Sequence (CES). Liquefaction induced damage affected 51,000 residential properties (Figure 1) with approximately 15,000 residential houses damaged beyond economic repair. The Earthquake Commission (EQC), a government insurer of private houses in New Zealand, funded a trial program to evaluate the efficacy of various cost effective ground improvement methods. The program objective was to investigate the technical viability of using ground improvement to reduce liquefaction vulnerability for the rebuild or repair of houses. The tested methods include rapid impact compaction (RIC), Rammed Aggregate Pier™ (RAP) reinforcement, driven timber piles (DTP), low mobility grout (LMG), resin injection, and shallow gravel and soil cement rafts.

Test panels for each ground improvement method were constructed at three sites in Christchurch in areas severely affected by liquefaction (Figure 1). The testing phase comprised pre- and post-improvement cone penetration testing (CPT) and crosshole shear wave velocity (V_s) testing, vibroseis T-Rex testing, and blast-induced liquefaction testing. The T-Rex shake test results and

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the blast-induced liquefaction test results are presented in van Ballegooy et al. (2015a) and Wentz et al. (2015) respectively. The blast induced liquefaction tests provided a relative assessment of the liquefaction susceptibility of unimproved and improved sites, which enabled a measure of comparison between the improved sites. Pore water pressure measurements made during blasting for the RAP-improved areas showed that excess pore pressure ratio (r_u) values were less than unity for sensors installed in both silty and clean sand materials. Although the numbers of measurements were not sufficient to be conclusive and the r_u measurements could be explained by other mechanisms (e.g., installations in thin sand layers, installations in layers not fully saturated), the measured site performance and the low r_u values resulted in a postulation that the installation of the RAP elements reduced the liquefaction susceptibility of both the clean sand and silty soil layers. The purpose of this paper is to describe the results of the measurements for the RAP treatment and to explore the mechanisms of RAP remediation.

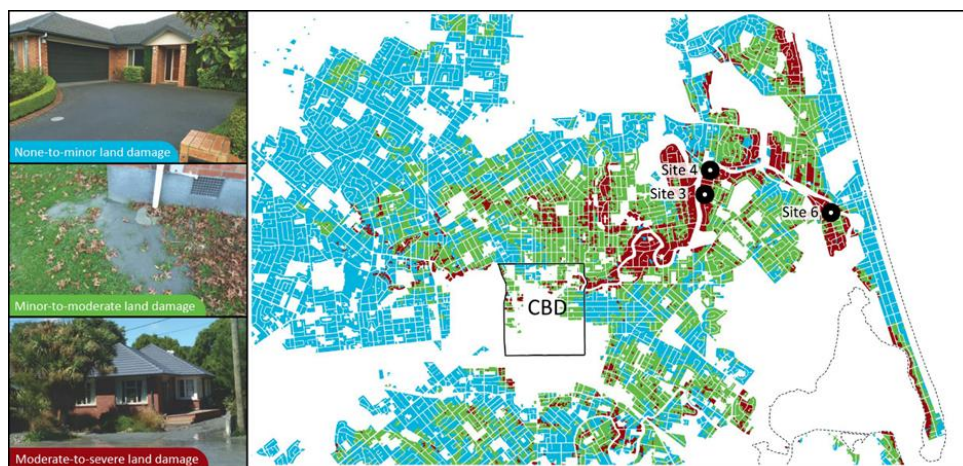


Figure 1. Severity and extent of the mapped liquefaction land damage on the residential land in Christchurch as a result of the CES. The white areas represent all the non residential land areas.

Rammed Aggregate Pier Ground Improvement Construction

RAP elements were constructed at the test sites 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 20 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 2).

Nine RAP test areas were constructed at three sites in Christchurch. The RAP elements were spaced 1.5 to 3.0 m on-center in a triangular pattern resulting in an area replacement ratio, A_r , ranging from 4 to 15%. The piers were installed to depths of 4 m in soil profiles that graded sequentially from sandy silt and silty sand to clean sand with depth. The siltier deposits were generally located within the top 1.2 to 2 m of the soil profile and the clean sands deeper than about 2 m. Groundwater was present at approximately 1 m below ground surface (bgs).

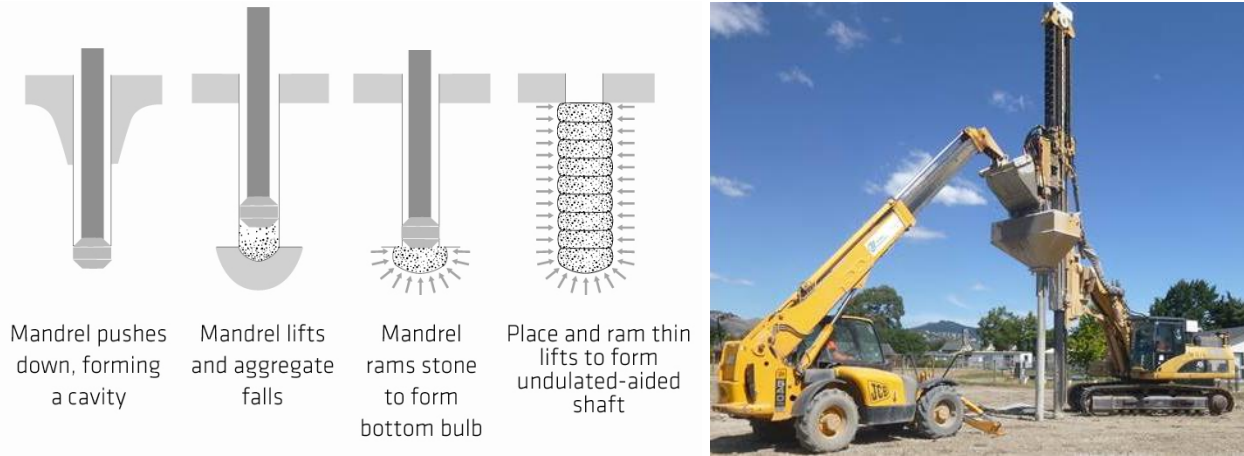


Figure 2. RAP ground improvement construction method applied at the three test sites.

At each of the tested areas, a series of in-situ CPT, crosshole V_S , and vibroseis investigations were conducted pre- and post-construction to quantify the improvement from the RAP installations. Figure 3 shows a schematic of the in-situ tests relative to the RAP locations. The CPTs were performed equidistant from the three installed RAP elements to conservatively measure the results at locations furthest from the piers. Crosshole V_S tests were performed in the nearby natural soil and in six of the RAP test areas where the pier spacing was 2.0 m ($A_r = 8\%$). As shown in Figure 3, crosshole V_S tests were made both between and across RAPs.

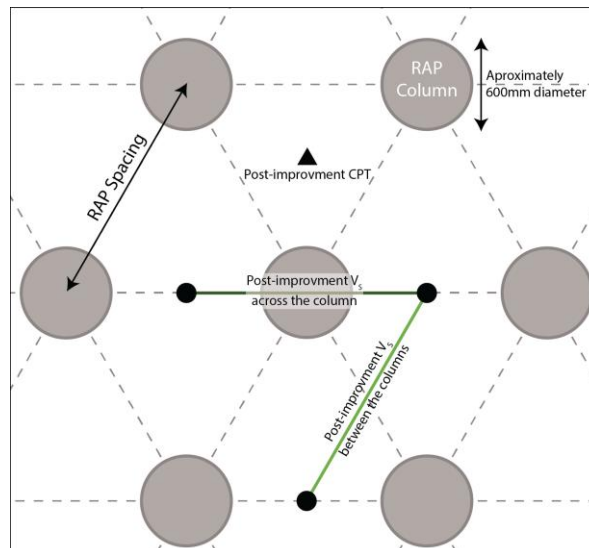


Figure 3. In-situ CPT and crosshole V_S test layout relative to the RAP locations.

CPT Investigation Results

Figures 4a and 4b show plots of all of the uncorrected pre- and post-improvement CPT tip resistance (q_c) measurements for three of the RAP test areas where the RAP spacing was varied. For additional clarity, Figures 4c and 4d illustrate the q_c values isolated for the soil layers with a soil behavior type index, $I_c < 1.8$ and $I_c > 1.8$, respectively. The data show that RAP installations consistently increase q_c in soil layers with $I_c < 1.8$ (i.e. lower fines content, FC) whereas a

minimal improvement in q_c occurs in soil layers with $I_c > 1.8$. These results indicate that soil layers with higher FC are not appreciably densified by RAP treatment, an observation that is consistent with those widely reported in the literature for other soil densification methods. Figure 4c and 4d also show three q_c envelope lines representing the computed q_c thresholds required to resist liquefaction for 25, 100 and 500 year return period ground motion cases for Christchurch as specified in the MBIE., 2014 guidelines computed using the Boulanger and Idriss (2014) liquefaction triggering method. Comparison of the pre-improvement q_c traces against the liquefaction triggering thresholds shows that the natural soils are generally predicted to liquefy between the 25 and the 100 year return period motions. This prediction is consistent with the observed performance of the ground through the CES. A comparison of the envelope of post-improvement q_c traces for the clean sand materials ($I_c < 1.8$) shows that the RAP improved soils are not predicted to liquefy under the Ultimate Limit State motions when the RAP $A_r = 15\%$, and are not predicted to liquefy for 85% of cases when the RAP A_r ranged between 5 to 8%.

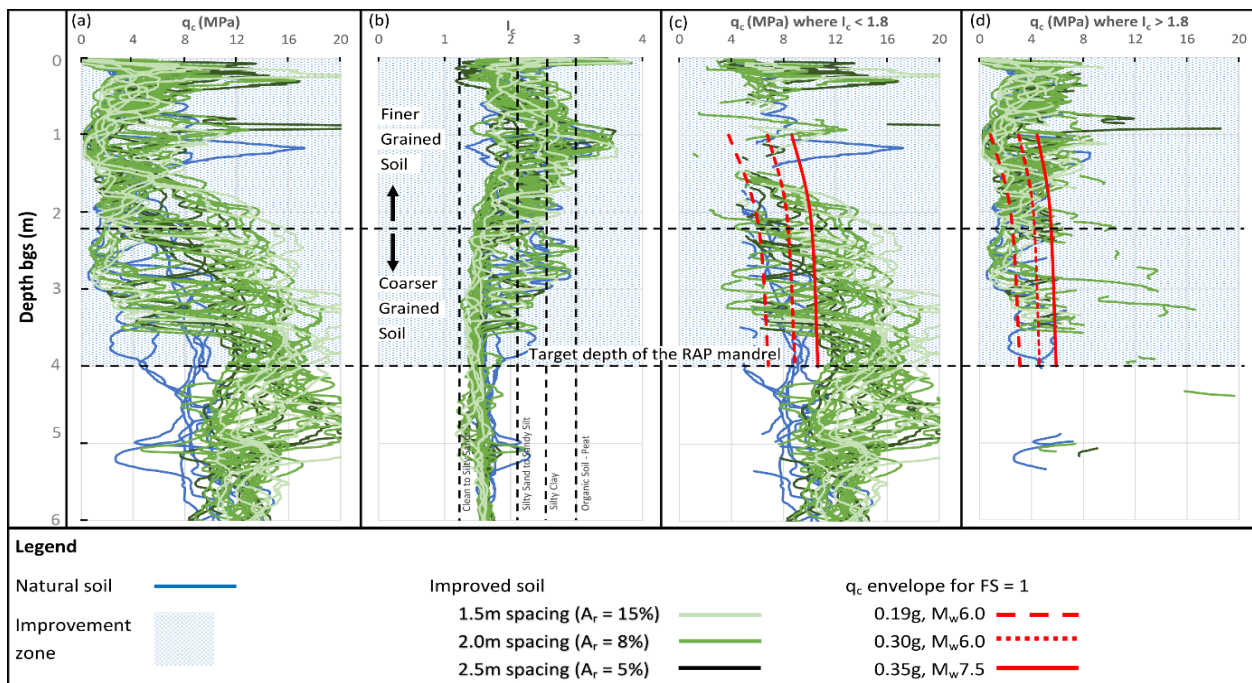


Figure 4. Pre- and post-improvement CPT traces of RAP improved soil at the three test areas: (a) q_c for all traces, (b) I_c for all traces, (c) q_c for all soil layers with $I_c < 1.8$, and (d) q_c for $I_c > 1.8$.

Figure 5a shows the average computed percentage increases in the Cyclic Resistance Ratio (CRR) values for various column spacings using the Boulanger and Idriss (2014) liquefaction triggering methodology. The data presented in Figure 4 indicate that for the silty soil layers ($I_c > 1.8$) there is a negligible increase in q_c , and hence CRR. Conversely, measured q_c and hence computed CRR (Figure 5a) values increase significantly for soil layers where the $I_c < 1.8$. As expected the greater measured and computed percentage increases occurred at higher A_r values.

The computed vertical one dimensional post liquefaction densification settlement, S_{VID} , and Liquefaction Severity Number (LSN) vulnerability parameter values (defined in van Ballegooy et al., 2015b) for all of the pre and post-improvement CPT traces at all of RAP test areas are shown in Figure 5b and 5c. The S_{VID} and LSN values are calculated for the MBIE (2014)

specified 500 year return period ground motions over the top 10 m of the soil profile using the Boulanger and Idriss (2014) liquefaction triggering methodology. The results presented in Figure 5 demonstrate a significant reduction in liquefaction vulnerability provided by shallow RAP ground improvement illustrating the benefits produced by RAPs for densifying clean sand and sand with silt. Because the q_c measurements were consistently carried out at the center of the RAP pattern where soil densification is less than at locations close to the RAP, the results may be considered to be lower bound conditions. Further, the computation method neglects any reduction in liquefaction potential that stems from the improved composite stiffness of the reinforced soil.

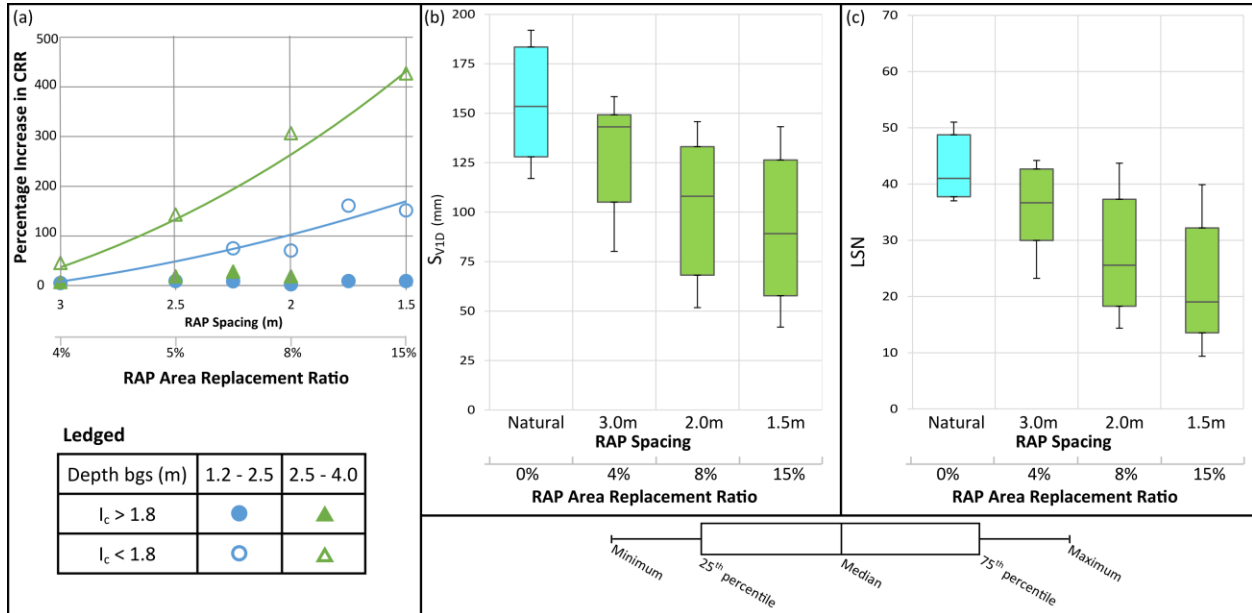


Figure 5. (a) Average percentage increase in computed CRR at different A_r for the RAP improved soils at the three test areas. (b & c) Distribution of the S_{VID} and LSN liquefaction vulnerability parameters at each CPT location for the natural and RAP improved soils.

Crosshole Shear Wave Velocity Investigation and Vibroseis Shake Testing Results

Crosshole V_S measurements were performed at six RAP panels where the pier spacing was 2.0 m ($A_r = 8\%$). Figure 6 illustrates the measured V_S and calculated small strain shear modulus, G_{max} , values for all six test panels. As shown in Figure 6, the crosshole V_S traces for the soil matrix between the RAP columns show a small but discernible increase relative to the natural unimproved ground for both the upper soil horizon with higher I_c values and the lower soil horizon with lower I_c values. Because G_{max} is proportional to the square of V_s , the increases in the G_{max} values shown in Figure 6b are optically more evident for both soil horizons.

The composite crosshole V_S (measured across the RAP elements) values are significantly larger than those for both unimproved and improved soil because of the presence of the stiff RAP elements in the measured results. In comparison with the natural soil, the average composite G_{max} values increased by approximately 15 MPa within the upper silty soil horizon and by approximately 65 MPa within the lower clean sand soil horizon. Unlike the CPT results that indicated negligible improvement in the siltier soils with $I_c > 1.8$, the clear improvement in G_{max}

suggests the potential for reduced liquefaction potential in the soils with higher FC.

Previous investigators (Baez and Martin, 1993; Priebe, 1995) proposed design methods that utilized the concept of composite shear strain stiffening for liquefaction mitigation. These methods considered a reduced shear stress demand on the native soil by considering shear stress attraction to relatively stiff reinforcing elements. The composite shear stiffness method was later subject to criticism (Goughnour and Pestana, 1998 and Rayamajhi, et al., 2012) or limitation (Girsang, et al., 2004; Green et al., 2008) because of the potential for flexural response of the reinforcing elements at high shear strain levels or because of the potential for gaps to occur between the soil and the reinforcing element for reinforcing elements constructed from rigid materials such as those used for piles, deep soil mixing, or jet grout columns. To provide insight into the prevalence of these mechanisms, the EQC performed high shear strain vibroseis testing at both reinforced and unreinforced areas at the test sites.

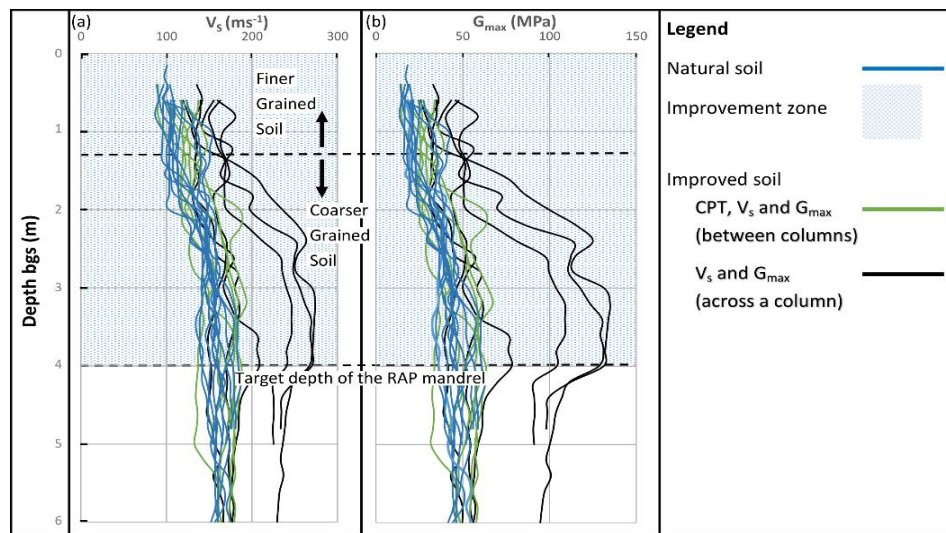


Figure 6. Pre- and post-improvement crosshole V_s (a) and G_{max} (b) profiles of the RAP improved soils at the three areas for RAP spacing of 2 m.

The vibroseis testing was performed by a team from the University of Texas, Austin by implementing the T-Rex mobile shaker to dynamically apply oscillating shear loads from the ground surface at both the unimproved and the improved soil conditions (van Ballegooy et al., 2015a). An array of geophones and pore water pressure transducers were installed in the ground directly below the T-Rex shaker to measure the cyclic shear strain, γ , induced in the ground and to measure the development of excess pore water pressures. Figure 7 presents both the previously-discussed V_s profiles and the measured peak shear strain profiles from the T-Rex shaker testing for natural soil and post-improvement RAP treated soil for two cyclic horizontal shear stress loading levels. Because the T-Rex shaker applies shear loads at the ground surface to a 2.3 m square plate, shear strains decay relatively rapidly with depth. The results shown in Figure 7 indicate that for each of the applied shear stress levels, the cyclic shear strains in the RAP reinforced soil were reduced to approximately 20% to 33% of the cyclic shear strain values measured in the unimproved soil, which indicates that the composite RAP reinforced ground is stiffer than the natural unimproved soil by a factor ranging from 3 to 5 at both low and high shear strain levels. The increase in the high-shear-strain composite stiffness decreases the

potential for development of excess pore water pressure and hence liquefaction triggering under cyclic loading.

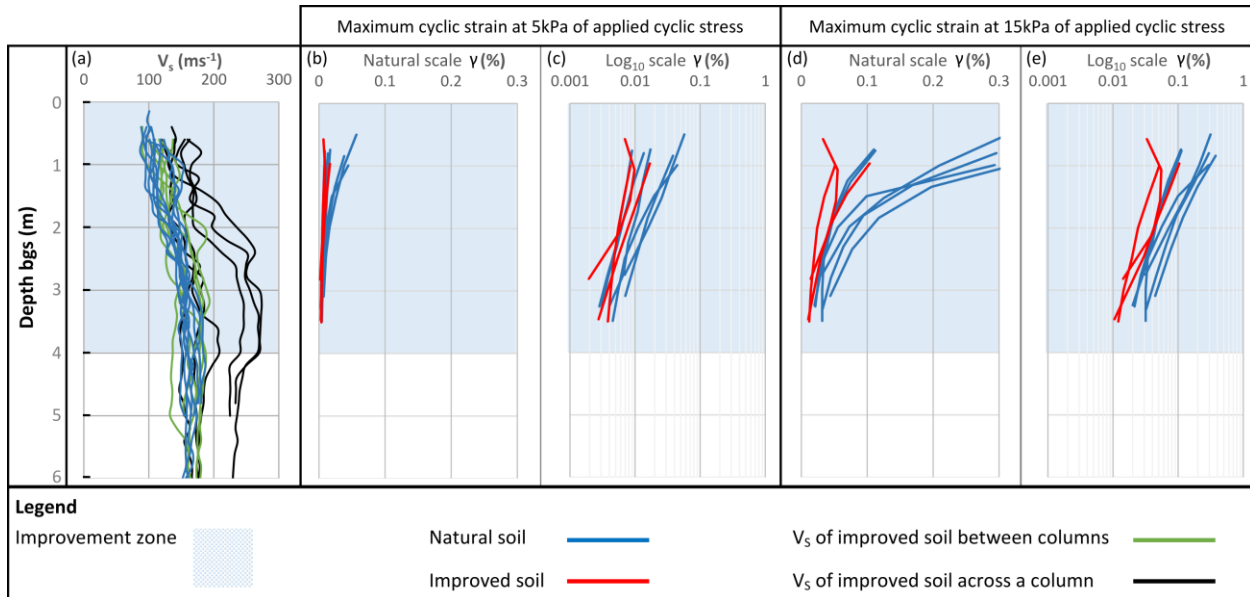


Figure 7. Pre and post-improvement crosshole V_s (a), γ at 5 kPa of horizontal cyclic stress applied by T-Rex at the ground surface (b & c), and γ at 15 kPa of cyclic stresses (d & e).

Discussion and Conclusions

The Christchurch testing program provided a unique opportunity to investigate the efficacy of a variety of ground improvement methods for mitigating soil liquefaction and provide insight into the mechanics governing the measured response. CPT q_c measurements confirmed that the RAP displacement method effectively densified clean sand deposits with $I_c < 1.8$ but did not provide measureable densification for the upper soil horizon with $I_c > 1.8$. Small strain crosshole V_s testing indicated an evident increase in the V_s measurements and corresponding G_{max} response for the improved natural soil and a large increase in V_s and G_{max} responses for the composite RAP-reinforced ground in both the clean and silty soil horizons. Large strain T-Rex testing showed that the composite reinforced ground within both the clean sand and silty soil horizons exhibited shear stiffness values greater than the unimproved soil by a factor of 3 to 5, confirming the effectiveness of reinforcing non-densifiable soil with RAP elements.

The results of the test program suggest that the improvement in the liquefaction resistance of the natural soil is related to the increase in the shear stiffness response of the RAP-reinforced ground. The increase in composite shear stiffness may be explained by a variety of mechanisms. It is likely that the uncemented RAP materials combined with the vertical ramming inherent in the RAP construction process results in a well-coupled pier-soil response that transfers shear stresses effectively across the soil-pier interface whereby the response is a byproduct of the unique construction process. It is also likely that the response results from the high lateral stresses that are applied to the natural soil during pier construction. These high lateral stresses serve to increase the mean stress conditions of the natural soil well above the normally-consolidated stress state (Handy and White, 2006), creating conditions that have been shown by

Harada et al. (2010) to increase CRR values within the reinforced soils. Regardless of the mechanism, the results presented herein show that the RAP elements consistently reduce liquefaction susceptibility in both clean and silty soils through a combination of soil densification and composite ground shear stiffening.

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