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ASSESSMENT OF THE DEGREE OF SOIL STIFFENING FROM STONE COLUMN INSTALLATION USING DIRECT PUSH CROSSHOLE TESTING

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Abstract

This paper outlines the results of a full-scale field testing program designed to assess the degree of soil stiffening resulting from the installation of shallow stone columns. Stone columns with similar area replacement ratios were installed in soils ranging from clean sands to sandy silts at a number of sites in Christchurch, New Zealand for liquefaction mitigation. Direct-push crosshole tests were performed before and after ground improvement using custom-built cone sensors to measure compression wave (V_p) and shear wave velocity (V_s) at 0.2 m depth intervals. Tests were performed across stone column to characterise the composite properties of the soil/column, and in-between stone columns to characterise the properties of the soil within the improved zone. Generally, the stone columns were effective at stiffening clean sand soil profiles, with the effectiveness of the technique shown to reduce in soils with higher fines content. Near the ground surface the stone columns were shown to be less effective at some locations, with the lack of confinement at these shallow depths preventing significant increases in stiffening, as indicated by V_s. A significant lowering of V_p was also identified at some sites after installation, indicating reductions to the degree of soil saturation. The change in the degree of saturation over time is a topic of interest for future study, as unsaturated soil conditions have the potential to increase liquefaction resistance.

Keywords: Stone columns; ground improvement; direct-push crosshole

1. Introduction

To assess the effectiveness of soil stiffening caused by installation of shallow vibro-replacement stone columns (SCs) for liquefaction mitigation beneath residential structures, a full-scale field testing program was undertaken in Christchurch, New Zealand [1]. This research was part of a wider research program to assess the effectiveness of a range of shallow ground improvement methodologies. These ground improvements were installed in soils ranging from clean sands to sandy silts at a number of sites across the city.

To assess the stiffening effect, compression wave (V_p) and shear wave (V_s) velocity measurements were made throughout the depth of the improved zone using direct-push crosshole tests with custom-built cone sensors containing a three-dimensional geophone array. Tests were initially performed before the installation of SCs to characterise the properties of the virgin soil. Following ground improvement installation, testing was carried out to characterise the properties of the improved zone, with a focus on both of the soil between the SCs, and the composite properties of the soil and SCs.

This paper provides an overview of the direct-push crosshole testing methodology, a relatively new methodology implemented in a number of projects across New Zealand. Results from two case study sites are presented in detail to provide an indication of the effectiveness of stone column installation in soils ranging from clean sands to sandy silts. The results from pre- and post-improvement CPT soundings used to assess the level of improvement of the soil between the SCs are also discussed.

2. Direct Push Crosshole Testing

Direct-push crosshole tests were performed to determine V_p and V_s as a function of depth using custom-built cone sensors containing a three-dimensional geophone array designed and constructed at the University of Texas. The geophones are housed in a stainless steel cone chassis with dimensions similar to a typical cone penetrometer test (CPT) tip. A source and a receiver sensor were advanced separately into the ground to the same depth using standard CPT rods and two small-scale cone penetrometer rigs. Data was acquired using a Data Physics ACE Quattro dynamic signal analyser connected to a laptop.

The test setup is shown schematically in Figure 1a, with the horizontal spacing between the source and receiver rods ranging from 1.5 – 1.9 m. This distance was defined based on the diameter of the column being tested, using a 500 mm offset on each side of the SC diameter in an effort to avoid areas of bulging on the side of the SC that would hinder advancement of the rods. At each test location the sensors were initially pushed to a depth of 0.4 m below ground level and the first test performed. The sensors were then both advanced at 0.2 m intervals to determine V_p and V_s down to at least 1 m below the depth of SC installation. CPT operators continually monitored the rods to maintain their verticality during testing. An example of the setup in the field is presented in Figure 2. In this example one CPT rod is advanced using a tracked CPT rig that is secured into the ground using augers, while the other CPT is advanced using a portable ram secured with augers.

At each depth testing was performed using a hammer impact source applied to the top of the source rod, with three separate impacts performed at each depth and stacked to increase signal-to-noise ratio. This impact develops compression waves (P-waves) that travel down the length of the source rod to its cone tip. The vertically oriented sensor (SV) at the bottom of the source rod (Figure 1a) was used to trigger the data acquisition system, eliminating the need to determine the P-wave travel time from the source impact at the top of the rod to the wave arrival at the cone tip. The P-wave reaches the end of the rod and creates both radially propagating P-waves and horizontally propagating, vertically polarised shear waves (S_{hv}-waves) at the base of the source that follow the path shown in Figure 1a. S_{hv}-waves, referred to as simply S-waves hereafter, were detected by the vertically oriented sensor in the receiver rod (RV). In a similar fashion, P-waves were detected by the horizontal geophone in the receiver rod that was oriented in line with the travel path (RH1). The other horizontally aligned geophone (RH2) is not used in these analyses.

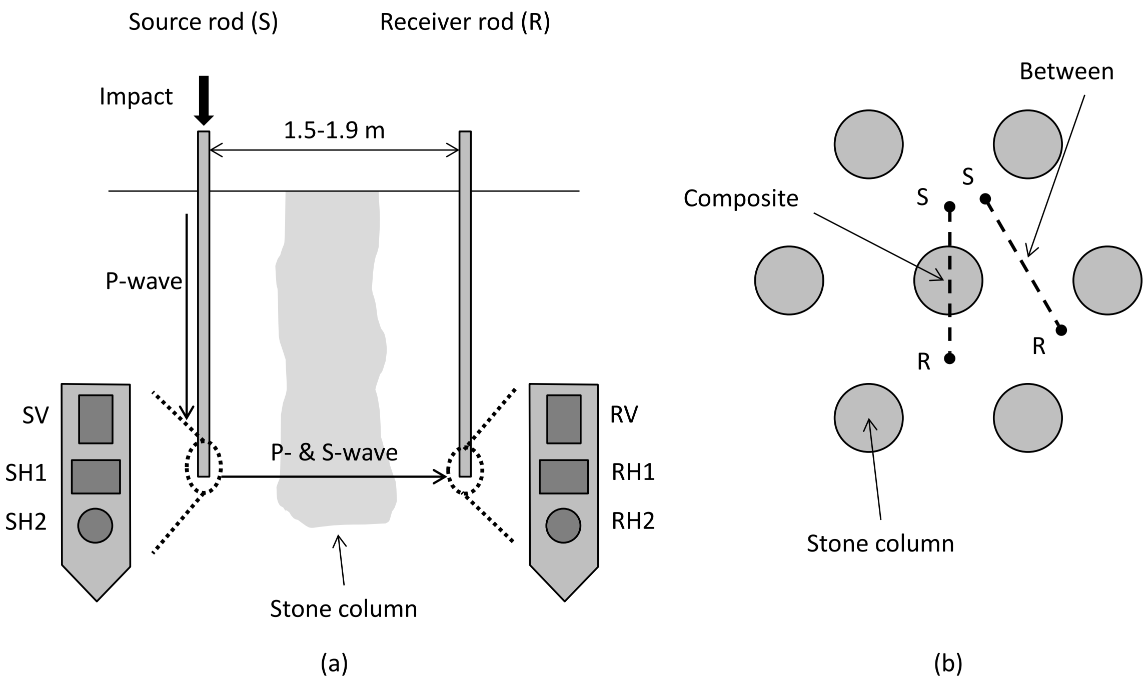

Fig. 1 – Schematic of the direct-push crosshole testing method a) elevation view, b) plan view

Fig. 2 – Photo of the setup of the direct-push crosshole method in the field, with track-mounted CPT rig to the left and portable CPT rig to the right.

The stacked/averaged waveform was used to identify the travel time (t_{RAW}) for first arrivals of direct compressive and shear waves at each depth. Waterfall plots that combine traces from each test depth into a single plot were used to more easily identify trends with depth. Picking of the first P-wave arrivals were fairly straightforward, while picking of S-wave first arrivals was sometimes complicated by a number of factors (such as refracted waves), and the first pulse in each trace of the waterfall plots was not always representative of the direct horizontal S-wave travel path. An example of a waterfall plot is provided in Figure 3, with examples of non-direct wave energy below the base of the improved zone (>6 m depth) in the S-wave data. Instead of picking the first arrivals, a later arrival is picked that is representative of the direct travel path through the soil at each test depth. Calibration testing was carried out to correct for the trigger delay time between the source and receiver rods, and this was used in combination with the picked travel time to calculate the velocity of each wave type. A detailed summary of the calibration and data interpretation methodology is given in Wotherspoon et. al. [2].

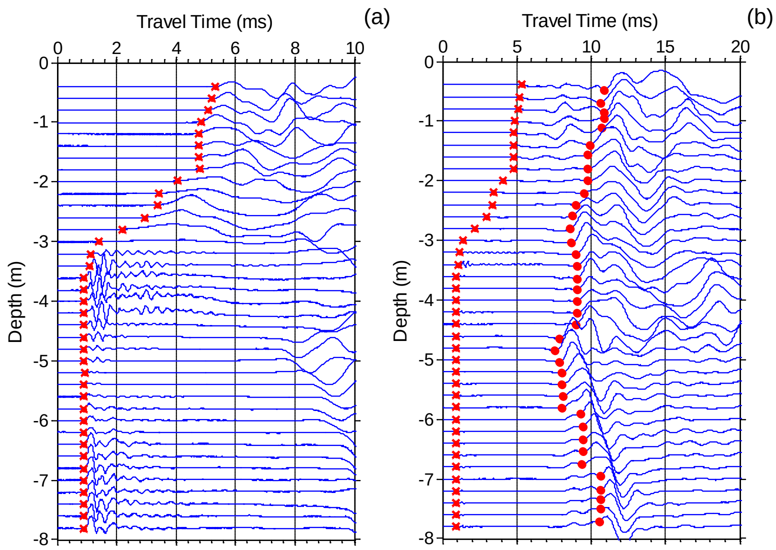

Fig. 3 - Waterfall plot a) In-line horizontal geophone records used to define compression wave velocity with P-wave picks (crosses); b) Vertical geophone records used to define shear wave velocity with S-wave picks (circles) and P-wave picks from a) (crosses)

One of the main shortcomings of the receivers used in these trial tests was the inability to measure the subsurface deviation of the source and receiver rods during testing. As stated above, precautions were taken to assure the CPT rods were initially vertical and deviated as little as possible in the soft soils during testing. Furthermore, the total pushing depths were typically less than 8 m, limiting the distance over which the rods could drift/deflect over. However, on occasion trends in the V_p and V_s with depth were used to identify and account for any deviation issues. Newer versions of the sensors, developed after the conclusion of this testing, have inclinometers installed adjacent to the geophones to allow tracking of the location of the tip of each rod. Experience from the use of these sensors has shown that the deviation over the top 8 m would not be significant enough to vary the results presented here by more than a few percent.

Tests were performed across each SC to characterise the composite properties of the soil and the SC, and in-between SCs to characterise the properties of the soil within the improved zone (Figure 1b). In-between tests were positioned in order to characterise the soil that was furthest away from the SC elements, representing the soil in the improved zone that was likely to be least affected by the installation of these elements (i.e. the least improved soil). Where possible, tests were performed prior to ground improvement or outside the improved zone to characterise the virgin (or unimproved) soil properties. Similar spacing between source and receiver rods were used for the composite, improved soil zone and virgin measurements at each test location.

3. Case Studies

All stone column ground improvements tested as part of this research project were constructed using a dry bottom feed method and river gravel-derived aggregate with at least two broken faces. A vibroflot is first vibrated/pushed down to the desired depth of improvement, then compressed air feeds aggregate though the vibratory probe and out through its base. As the vibroflot is partially withdrawn, aggregate is delivered into the resulting void and the weight of the vibroflot is applied onto the aggregate in combination with a vibratory load. This process is performed in stages as the probe is progressively withdrawn towards the ground surface to form a column. This method is used to densify both the aggregate and the surrounding soil.

The details of the three case studies presented in this paper are outlined in Table 1. Case study X and Y are presented in detail, with data from case study Z used in aggregated comparisons in the Discussion section of this paper. The nominal diameter of the SC, the design spacing, pattern and installation depth, and the average area replacement ratio (ARR, equal to the ratio of the total SC cross sectional area and the total improved area) is summarised for each site. For each case study composite tests are denoted as C, between tests as B, and virgin soil tests as V, with the number of each test also indicated (i.e. C1 for composite test 1).

The installation characteristics of each site are not the same, therefore we are not intending to comparing results from site-to-site. Instead, the aim is to identify general trends in the performance of the SC installation for different soils. The focus of this study is the stiffening effect of stone column installation, as indicated by changes in V_s or G_{MAX}, with other potential benefits, such and increased lateral stress and drainage effects, not discussed in this paper.

Table 1 - Summary of case study ground improvement details

Site	SC Diameter	Spacing	Depth	ARR	Pattern
X	0.68 m	1.67 m c/c	4.0 m	15.0%	Triangle
Y	0.88 m	2.14 m c/c	4.8 m	15.3%	Triangle
Z	0.905 m	2.14 m c/c	4.5 m	16.4%	Triangle

3.1 Site X

Figure 4 summarises the soil profile data from crosshole (V_p and V_s) and the pre- and post-improvement CPT testing data (tip resistance q_c and soil behaviour type index I_c [3]) at Site X, with the depth of SC installation indicated by the shaded region. The water table from CPTu data was at a depth of 1.9 m. At this site the cross-hole testing was performed 11 days after SC installation, while the CPTs were performed 18 days after installation. The horizontal spacing between source and receiver was approximately 1.7 m at this location, therefore for composite tests approximately 40% of the travel path was through the SCs.

Particle size distribution data at this site are summarised in Figure 5, with fines contents (FC, % passing 0.075 μm sieve) of 1-4% present throughout the soil profile. The soil behaviour type index from CPT data compares well with the measured fines content values, indicating clean sand – silty sand below a depth of 1 m. The particle size distributions indicate poorly (uniformly) graded fine sands at this location, the characteristics of which fall into the range of highly liquefiable soils as defined by Tsuchida [4].

Figure 4 shows there is a clear increase in V_s within and below the column installation depth for all tests (both across/composite C and between B) compared to the virgin soil measurements. It is notable that there is little difference in the C (composite) and B (between) V_s measurements, suggesting that the stiffness of the SC elements are not too dissimilar from the stiffness of the surrounding improved soil. Overall, the SC installation increased V_s by approximately 50 m/s throughout the improved zone, a 70-100% increase in G_{MAX}. The V_p measurements are dominated by the degree of soil saturation, and indicate a slight reduction in the depth to saturation post-installation over most of the improved zone. In the virgin measurements there is a drop in saturation at approximately 3 m depth, reaching saturation again at 3.5 m depth. Interestingly, the water table depth defined from CPTu data does not match the depth to saturation indicated by the virgin cross-hole test data (V_p = 1500 m/s). We have consistently noted this trend across much of Christchurch, meaning the depth where the soil reaches full saturated is often 1-2 m below the depth of the hydrostatic water table.

Pre- and post-improvement CPT soundings were also able to assess the level of improvement of the soil between the SCs. Figure 4 shows that below a depth of 1 m there is a clear increase in tip resistance following SC installation, with 100-300% increase in q_c. Approximately 1 m below the base of the improved zone the pre-and post-improvement tip resistance have similar values, indicating that the SC installation has little effect below this depth. This is also evident in the V_s measurements, with all post-improvement shear wave velocities similar to the virgin values at approximately 2 m below the base of the SCs.

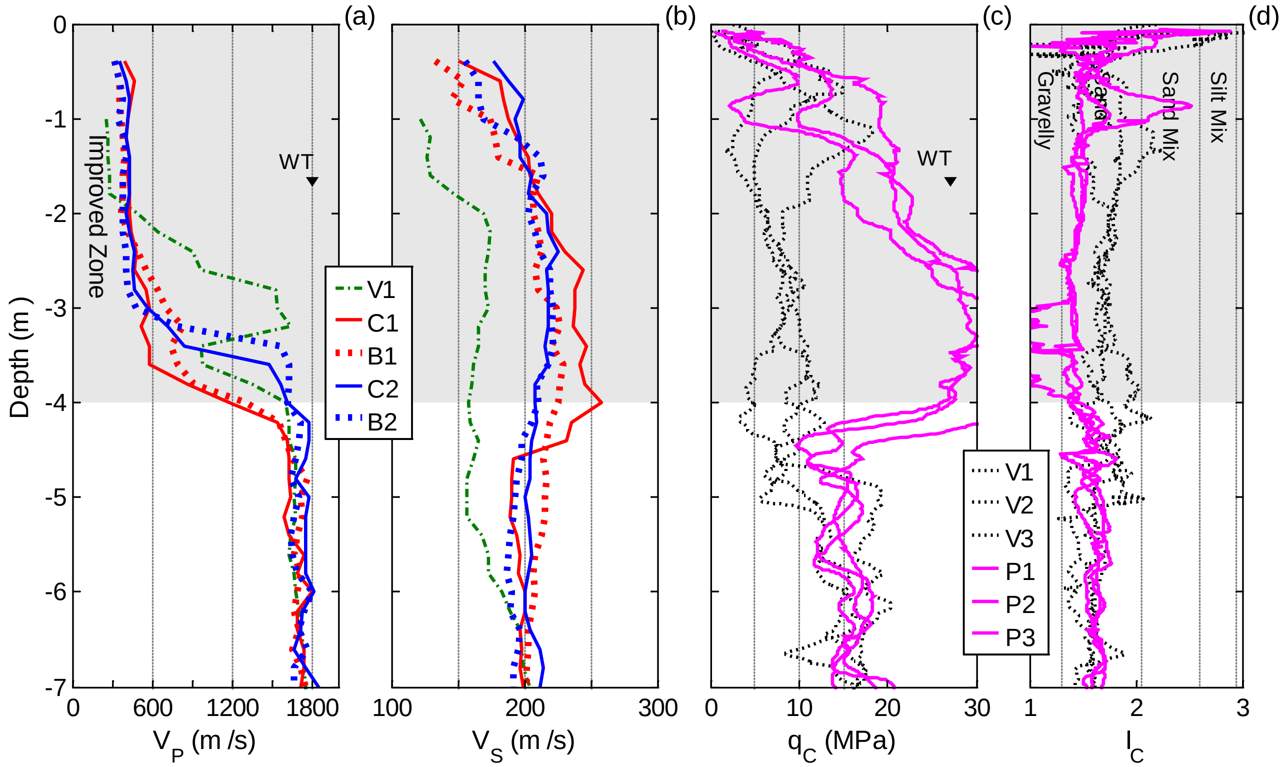

Fig. 4 – Site X characteristics: a) P-wave velocity; b) S-wave velocity; c) CPT tip resistance; d) soil behavior type index. V1-V3 denotes pre-improvement CPT data and P1-P3 denotes post-improvement data.

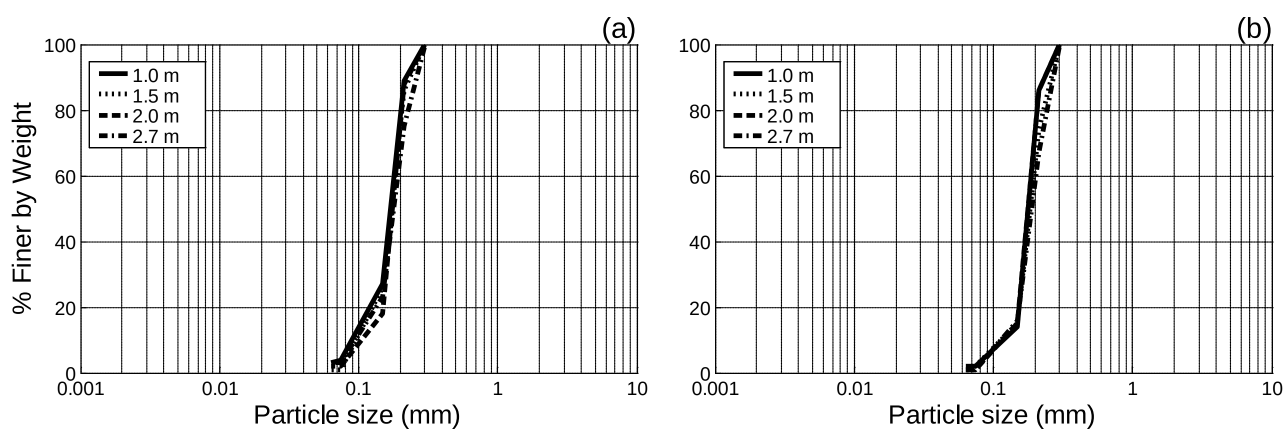

Fig. 5 – Particle size distribution characteristics for Site X: a) Location 1; b) Location 2

3.2 Site Y

Figure 6 summarises the Site Y soil profile data, with the depth of SC installation indicated by the shaded region. The water table from CPTu data was at a depth of 0.9 m. At this site the cross-hole testing was performed 23 days after SC installation, while the CPTs were performed 39 days after installation. The horizontal spacing between source and receiver was approximately 1.9 m at this location, therefore for composite tests, approximately 46% of the travel path was through the SCs.

The soil profile was variable at this site, as evidenced by the pre-improvement CPT q_c and I_c in Figure 6. Above 2.5 m depth the site has similar characteristics, with low q_c values in sandy silt and silt deposits. Below this depth there is variability, with part of the site having similar deposits extending below the improved zone. In other parts of the site the soil profile transitions to a cleaner sand material, with q_c increasing up to 15 MPa. Crosshole tests C1 and B1 were carried out in the siltier soil profile (denoted Area 1), with tests C2 and B2 located in the profile with cleaner sand at depth (denoted Area 2).

Particle size distribution data at this site are summarised in Figure 7. Above 2 m depth, in the most silty soils, FC of 99% were measured, representative of the soils across the entire site. FC's of 69% and 93% were measured at depths of 3.8 and 4.5 m, respectively (Figure 7b), representative of the silty nature of Area 1 of the site. A FC of 24% was measured at a depth of 2.65 m (Figure 7b), which is also representative of the soils across the site at this depth, with the I_c values for both pre-improvement CPT soundings in Figure 6d similar at this depth.

The crosshole measurements at Site Y are summarised in Figure 6a and b. The virgin soil measurements are representative of the characteristics across the entire site above 2 m, however below this depth the measurements are likely more representative of Area 1. Above 2 m depth, in the most silty soils (typically 99% FC), the SC installation resulted in no stiffening of the ground compared to the virgin conditions, with the V_s in the improved zone equal to or less than the virgin soil V_s. Below 2 m the performance was variable, with little-to-no improvement in V_s for tests B1 and C1, and an increase in V_s for tests B2 and C2 from 25 – 100 m/s relative to the virgin soil. Once again, there is little difference in the C and B V_s measurements throughout much of the improved zone. These variable results suggest that the stone column installation was not able to effectively stiffen the sandy silt and silt at this site, with the degree of improvement also varying spatially across the site due to soil variability.

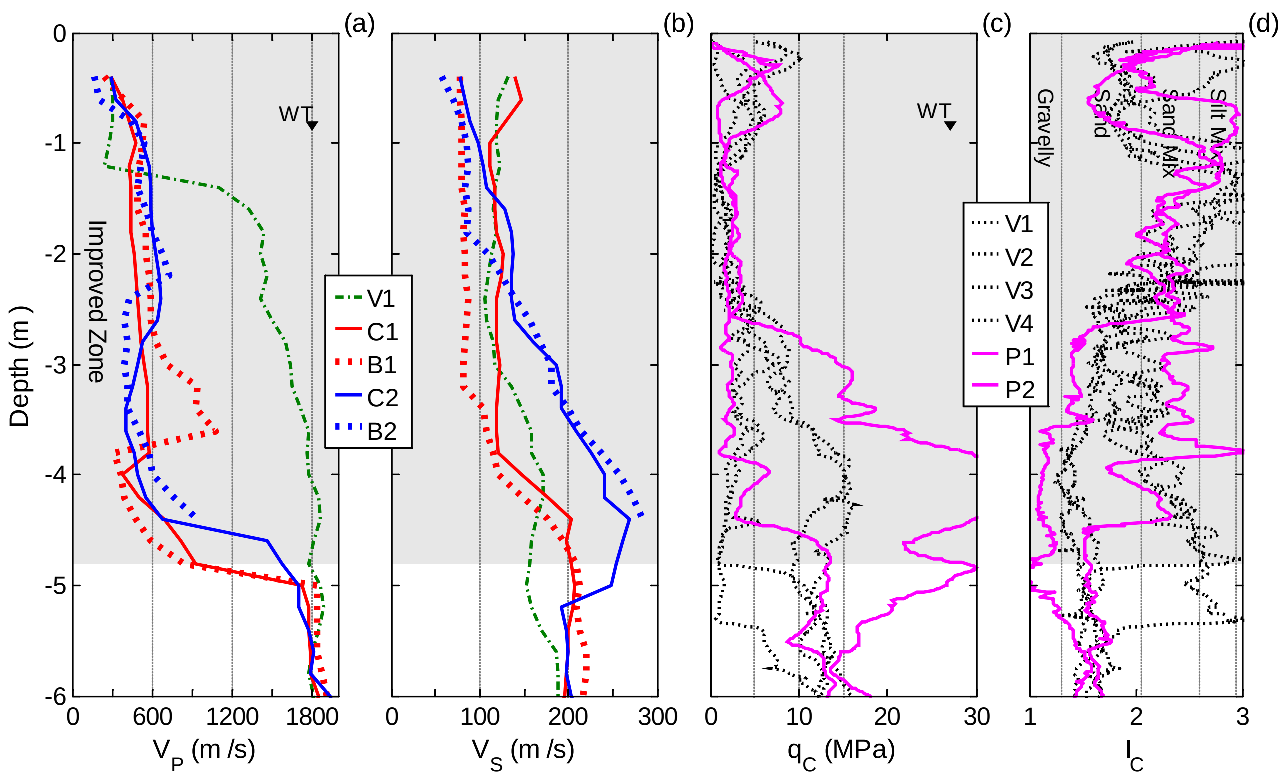

Fig. 6 – Site Y characteristics: a) P-wave velocity; b) S-wave velocity; c) CPT tip resistance; d) soil behaviour type index. V1-V4 denotes pre-improvement CPT data and P1-P2 denotes post-improvement data.

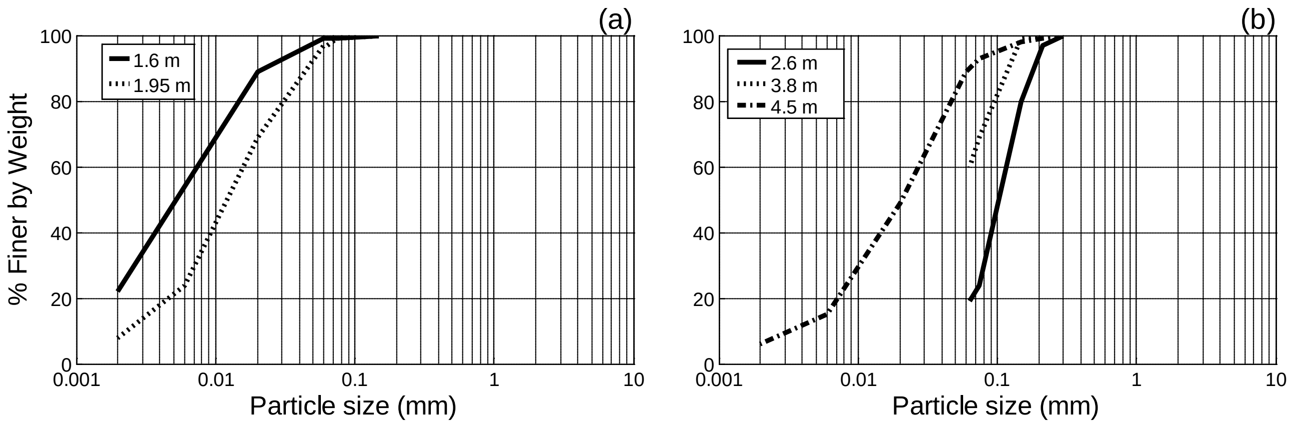

Fig. 7 – Particle size distribution characteristics for Site Y: a) 0-2m depth, representative of the characteristics of the entire site over this depth range; b) Below 2 m depth, with 3.8 and 4.5 m samples representative of Area 1 and 2.6 m samples representative of both Area 1 and Area 2 at this depth.

The V_p measurement show the dramatic reduction in saturation at the site, with V_p reducing to approximately 600 m/s throughout much of the improved zone depth. Below the improved zone the soil quickly becomes saturated again, with pre- and post-installation measurements both indicating saturation. At Site X the bottom metre of the improved zone was saturated post-installation, while at Site Y there is still clear de-saturation. The differences in these characteristics at the two sites may be due to the higher fines content of the soil layers at Site Y, slowing the seepage rate of water from the surrounding area into the improved zone.

Pre- and post-improvement CPT soundings were also able to assess the level of improvement of the soil between the SCs in the two areas at the site. Figure 6c shows that above 2 m depth there is little difference in the pre and post q_c values, similar to the V_s values described above. In Area 1 there is little difference in q_c below 2 m depth, with some evidence of a reduction in q_c from 2-4 m depth. In Area 2 there is a significant increase in q_c below 2 m depth due to SC installation, mirroring similar improvement indicated by the V_s measurements.

4. Discussion

The results presented here reinforce generally recognized findings that vibro-replacement stone columns are effective at increasing the V_s of cleaner sand sites, with the stiffening effect reducing in soil profiles with high fines content. Figure 8 provides a summary of the degree of improvement of the soil within the improved zone compared to the fines content at the three case study sites outlined in Table 1. The locations of samples taken for particle size distribution testing and post-installation crosshole testing are both in close proximity to one another, and data above 1.5 m depth is not included due to the influence of lack of confinement at these shallow depths. Figure 8a shows the relationship between the virgin soil V_s and fines content, with no clear relationship between these two factors at the case study sites, with V_s of between 100 and 170 m/s across the range of soil types.

The degree of improvement (or weakening) resulting from stone column installation is shown in Figure 8b using the difference between the post-installation V_s and the virgin soil V_s at each depth where fines content laboratory data was available. Interestingly, for fines content ranging up to approximately 30% the SC appear to be quite effective at stiffening the soil, with increases in V_s of 40-80 m/s and no clear correlation between fines content and level of stiffening. Furthermore, there is also no clear separation between the increase in V_s across SC and between SC measurements in this range, with the between measurements less than the across at some sites, and higher at others.

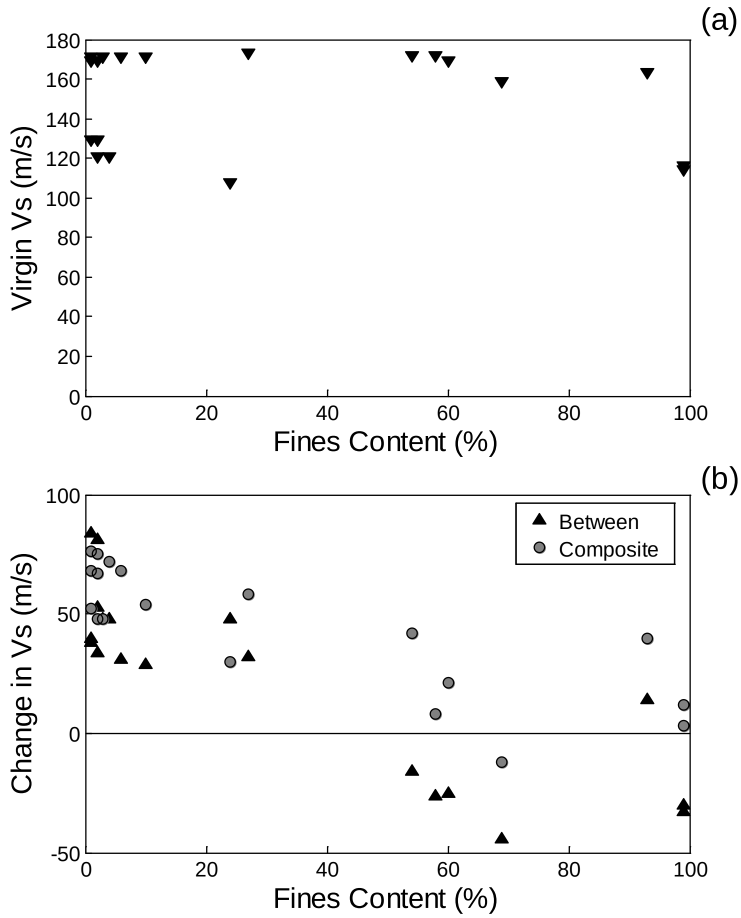

Fig. 8 - Comparison of a) virgin soil V_s and fines content; b) change in V_s and fines content at stone column installation sites.

Between 30-50% fines content there is a gap in the data, but above a fines content of 50% the data indicates a reduction in the effectiveness of SC installation as FC increases, with this trend most evident when focussing on the between SC data. For these between SC measurements a negative effect due to installation is evident at all but one of the locations with a FC greater than 50%, with reductions in V_s of 10- 50 m/s. For the locations with a FC less than 30% the smallest increase in V_s for the between SC measurements was 25 m/s, clearly demonstrating the improved performance of SC installation in soil with lower fines content. A larger dataset, especially in locations with a fines content of 30% and above would be useful to further inform the effectiveness of SC installation across a wide range of soil conditions.

It should again be noted that at all these case study sites an ARR of 15-16.4% was used. If different ARR specifications were used the effectiveness of SC installation would also be likely to change. This highlights that in order to ensure that the desired ground improvement characteristics are achieved, verification testing is important when using SCs, especially in silty soils and locations with variable ground conditions, so that modifications to the methodology can be made (such as increasing the area replacement ratio) when the desired results are not achieved.

The reduction in the degree of saturation was a characteristic evident at a number of other sites where SCs were installed (Tonkin & Taylor 2014). This is most likely a result of the use of compressed air during the SC installation process at these sites. If compressed air is not used, as is the case in some SC installation methods, it is unlikely that this de-saturation effect would be so prominent. Comparison of SC and rammed aggregate piers (RAP) installation by Wotherspoon et al. [2] showed that there was no de-saturation effects following the installation of RAP elements, a method that does not utilise compressed air during installation (along with other differences in installation methodology that are not the focus of this paper).

While the crosshole method is useful for SC assessment, the CPT tip resistance has also been shown to be able to indicate improvement of the soil in the improved zone surrounding the SC elements. However if a measure of the stiffness contribution of the SC elements themselves is desired, the crosshole method seems to be the most effective approach. Along with this, the ability to measure the degree of saturation through P-wave velocity measurements is another advantage of the crosshole method over traditional site investigation methods.

5. Conclusions

This paper outlines the use of direct push crosshole testing to assess the effectiveness of soil stiffening caused by installation of shallow vibro-replacement stone columns in Christchurch for liquefaction mitigation. Case study results highlighted the usefulness of the crosshole technique, as it can reliably demonstrate the stiffening effect of the composite soil-improvement mass. Although more traditional post-construction verification testing methods can identify the improvement of the soil within an improved site, they cannot be used to infer the composite stiffness of the SC and soil.

Very similar composite and between column V_s measurements were evident at each of the case study sites presented here, indicating that the V_s of the SC elements may not be significantly higher than that of the surrounding soil. Results from sites where the soil profile is variable laterally demonstrate that in order to ensure that the desired ground improvement characteristics are achieved, verification testing is important, especially in very silty soils, so that modifications to the methodology can be made when the desired results are not achieved.

A lowering of V_p was also identified at many SC sites after installation, indicating reductions to the degree of saturation, often throughout the entire improved zone. This change in the degree of saturation and its stability over time will be of interest for future study, as it may have an impact on the triggering of liquefaction.

6. Acknowledgements

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