The paper was published in the proceedings of the 12th Australia New Zealand Conference on Geomechanics and was edited by Graham Ramsey. The conference was held in Wellington, New Zealand, 22-25 February 2015.

Experimental study of driven pile capacity improvement due to compaction grouting

S. Samsuddin[1], S. Wang[1], S. Sloan[1]

[1]ARC Centre of Excellence for Geotechnical Science and Engineering, The University of Newcastle Callaghan, NSW 2308, Australia; email: sufyan.samsuddin@uon.edu.au, Shanyong.Wang@newcastle.edu.au, Scott.Sloan@newcastle.edu.au

ABSTRACT

Loose sand is likely to be compressible. It also might have insufficient strength to support large loads on driven piles. The conditions should be improved by densification of soil near the bearing end of the pile. A new laboratory setup for investigating the effect of compaction grouting on pile capacity was designed and assembled. This apparatus allows a model pile to be driven into a sample and then compaction grouting is delivered through the wall of the pile. The pile penetration resistance, grout injection pressure, soil pore pressure and, the void ratio change of soil are monitored. These parameters are experimentally studied for its role in the pile capacity improvement. In addition, estimation is made to the contribution of the injected volume to the pile capacity. The result indicates that compaction grouting could increase the pile capacity by 40 percent. In addition to that an increase in compaction grouting pressure has been shown to increase compaction efficiency. Compaction grouting should be a useful method in improving driven pile capacity.

Keywords: loose sand, compaction grouting, driven pile

1 INTRODUCTION

Engineering advancement in the past has allowed engineers to improve ground conditions for construction works. However human expansion and environmental protection have caused engineers to work on more poor and unstable soils. As more construction work moves to the seabed, ground improvement becomes more difficult. Instead, engineers rely on anchoring structures on the seabed using methods such as piling. Driven pile capacity relies partly on compaction achieved at the bottom of the pile during pile driving works especially in loose granular materials such as poorly graded sand and silts. The pile's skin may need to be improved against negative friction generated by soft soil. In such a case where the pile is expected to be driven through soft ground and founded on loose sands, compaction grouting may provide an alternative approach to piled foundation construction. Compaction grouting is traditionally done by injecting viscous soil-cement mixture with particle sizes sufficient to be mobile and yet forms a growing homogeneous mass as pressurized injection continues without permeating the soil pores in order to control compaction of loose soils (Bruce 2005).

Many researchers have constructed physical models to study the impact of compaction grouting on piled foundations. For example, Mutman et al. (2012) and Fang et al. (2013) made a field study of drilled shaft improvement by compaction grouting. Pooranampillai et al. (2010) on the other hand managed to provide a more controlled environment by studying the impact of compaction grouting on capacity of drilled shaft in a steel chamber. However all of these investigations were limited to non-driven piles with grout delivery system near the bottom of the pile. It may be problematic to deliver compaction grouting at the tip of driven piles since the pile driving action could block or damage the delivery hole near the pile tip. Also, the pile driving test creates disturbed soil zones unique only to driven piles. In the current laboratory tests, the model pile could be driven and the model grout is delivered through the wall of the pile (Figure 1a). This investigation focuses on compaction grouting in loose poorly graded Stockton Beach sand. The laboratory setup of the new apparatus is presented herein. Pile penetration resistance are measured to compare the pile capacity and amount of densification with and without compaction grouting.

2 LABORATORY SETUP

Figure 1b shows the systems used for pile driving, compaction grouting and the pile test. It is made of a standard large 1000kPa triaxial cell connected to pressure/volume controllers to control the cell pressure, the back pressure and the injection pressure.

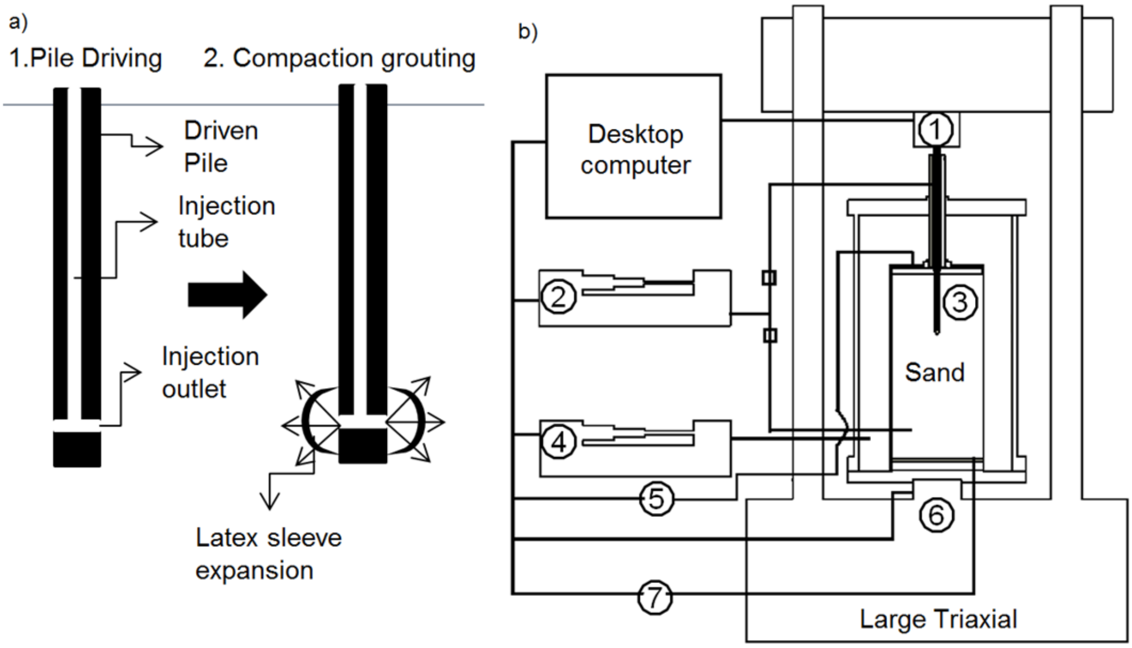

Figure 1. a) Compaction grouting of driven pile b) Schematic layout. Notes 1: Load cell, 2: Injection/Back pressure/volume controller, 3: Model pile (Full penetration), 4: Cell pressure/volume controller, 5: Volume gauge, 6: Displacement piston, 7: Pore pressure transducer

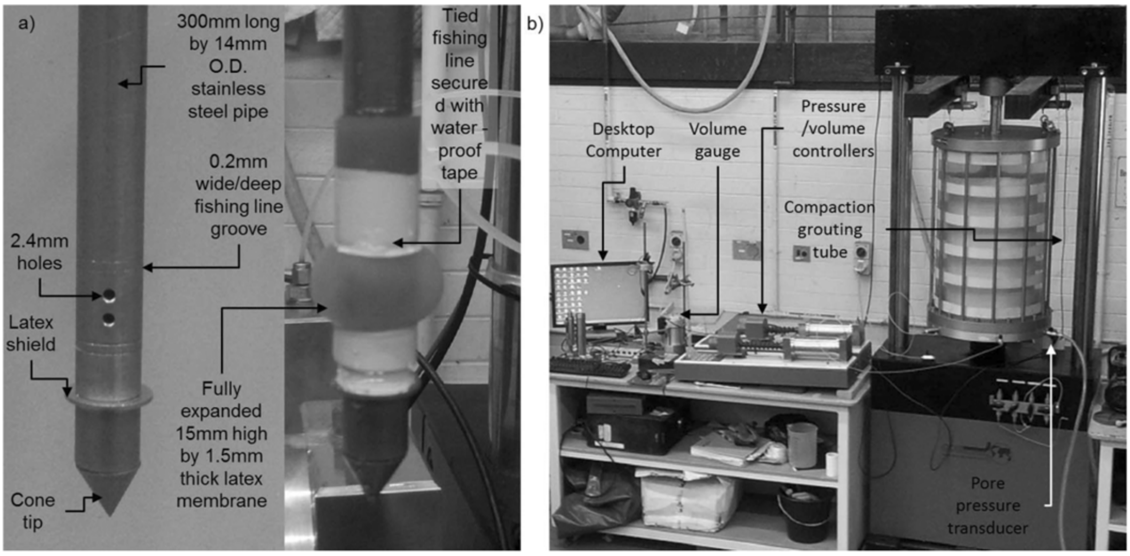

Figure 2. a) Model pile and grouting system showing injection port, b) Part of the laboratory test setup.

The change of the specimen volume is measured concurrently using the cell pressure/volume controller connected to the cell and a volume gauge connected to the drained top of the sample.

The model pile is placed at the top of the sample with its tip through the sample top cap and embedded into the centre of specimen. The sand sample has 0.3m diameter and it is 0.6m high. The model pile is driven into the sample by pushing the sample upwards as the top of the pile bears onto a load cell. The triaxial cell displacement, the pressure/volume controllers, the volume gauge and the pore pressure transducer are all connected to and controlled by a desktop computer.

The compaction grouting system in Figure 2a is prepared by rolling a standard 12.7mm outer diameter medical latex tube over the model pile. Fishing line is then used to tie the tube onto the stainless steel model pile over the prepared grooves. Waterproof tape over the fishing line is used to prevent the unravelling of the fishing line during experiment. A stainless steel washer is provided to protect the membrane during pile driving. The latex membrane technique has been deployed by other researchers to model compaction grouting (Au et al. 2006). However unlike Au et al (2006), Pooranampillai et al (2010), Mutman et al (2012) and Fang et al (2013), the injection location and direction in this new apparatus differs. In this new apparatus, the grout is injected sideways to expand the grout mass laterally instead of the usual downward injection done by past researchers. During injection, the pressure or volume is measured and controlled using the pressure/volume controller. The resultant compaction is then measured by volume gauge connected to the top of the sample. Figure 2b shows the laboratory test setup.

In preparation of the test, two processes must be completed. Firstly, the sand sample must be prepared; secondly the model latex membrane must be calibrated before the pile is assembled on top of the sample. In this test the sand sample is prepared by raining the sand sample into the sample former that sits on top of the triaxial cell base. The dry pluviation techniques used a drop height of 600mm onto the top of the sample. Sand has to pass through a disc filter at the top of the drop height. The filter is 300mm in diameter and has 8mm holes spaced in staggered patterns over centre to centre distance of 40mm. This particular filter is used to prepare loose sample. For samples at other densities, different size holes are used. Once the raining volumetric speed is control using the filter the sand immediately drops onto a 236micron sieve before free falling a minimum distance of 600mm into the sample former. The properties of this Stockton Beach sand have been reported by other researchers (Ajalloeian, 1996) as summarised in Table 1. It is classified as poorly graded silty sand in the Unified Soil Classification System (USCS). The density of the prepared sample is almost the same lying between 1500 and 1550 kg per cubic metre (Figure 3a).

The model latex membrane is calibrated by expanding the latex membrane in mid-air as in Figure 2a by controlling the injection volume and monitoring the pressure needed to create the volume. Several expansion and deflation cycles are done to ensure that the latex membrane behaves similarly under repeat expansion. As shown in Figure 3b the latex membrane expansion resistance for a particular injection volume is consistent for different latex medical tubing samples used.

3. EXPERIMENTAL PROCEDURES

The experiments are done in five stages. In the first stage the sample is wetted with de-aired water. De-aired water is prepared using a de-aerator with 100kPa vacuum for a minimum of 20 minutes. The sample is also kept under 30kPa vacuum pressure connected to the top of the sample. Once the de-aired water is ready, the de-aerator is connected to the bottom inlet of the sample. The de-aerator is then exposed to atmospheric pressure so that the atmospheric pressure is pushing the water into sample under 30kPa pressure gradient. The wetting process is stopped once visual inspection has shown that no trapped air is visible on the sample membrane and the discharged water at the top outlet has no more air bubbles. The water inlet and the drain outlet are then closed in order to conduct back saturation.

The back saturation process is done by increasing back pressure and cell pressure at the same rate but with a difference of 40kPa between both pressures. The back volume is monitored to ensure water is not exiting the sample during the process. The pressure increase is stopped once the cell pressure reached 960kPa or once back volume is measured exiting the sample. The sample is then kept at constant back volume and constant cell pressure for at least 6 hours. After that the cell pressure is reduced to 40kPa and a B value test is conducted up to 100kPa. The process is repeated until the Skempton B value is above 0.95 and does not change significantly over time.

In the second stage, the sample is kept at 100kPa confining pressure. The back pressure controller is then transferred to the drainage outlet at the top of the sample to maintain a constant back pressure of 8kPa. The sample is then lowered down so that the model pile can be configured to bear on the load cell (Item 1 of figure 1b). The cell pressure is monitored for stability. Once that is achieved, the third stage starts.

In the third stage, the sample is pushed onto the model pile by extending the displacement piston (Item 6 of figure 1b) at a constant rate of 0.1mm/s. During this stage, load cell force value and the back pressure volume change is recorded to measure pile penetration resistance and also sample volume change due to pile penetration. The penetration distance is kept around 70 to 80mm before the sample is lowered again for the repetition of stage three. Stage three is completed once the total penetration is around 250mm and the top cap is connected again to the load cell.

Table 1: Soil properties of Stockton Beach sand (Ajalloeian,1996)

Test	Log bulk modulus, κ	Minimum void ratio, e_{min}	Max void ratio, e_{max}	Angle of friction (Degrees), φ	Dilation angle (Degrees), ψ	Particle size distribution					
						D10 mm	D30 mm	D50 mm	D60 mm	Cu	Cc
Loose sand sample	0.0374	0.49	0.78	30 to 35	0 to -5	0.24	0.36	0.4	0.41	1.71	1.32

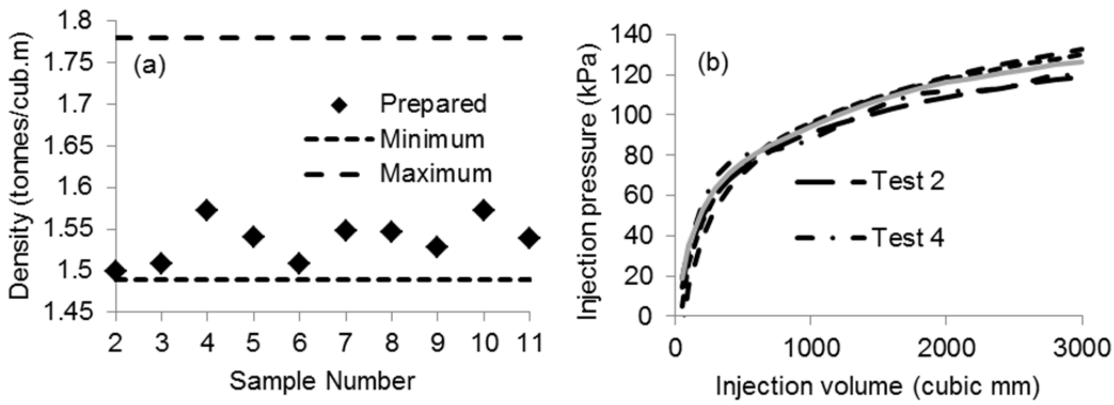

Figure 3. a) Repetitiveness of sample preparation method; b) Repetitiveness of latex membrane expansion

Compaction grouting is done in stage four. Back pressure control is transferred to the injection tube to control compaction grouting and pressure. The sample top drainage is then connected to volume gauge to measure sample volume change during the compaction process. Cell volume is also recorded to double check the volume gauge readings. A series of injection pressure increase and rest period is introduced until the injection pressure reached 600kPa or the injection volume exceeds 3000 cubic millimetre. An example of the injection pressure pattern is shown in Figure 4. The actual pressure experienced by the soil should be less since the latex membrane also tries to resist the membrane expansion. By knowing the injection volume, the actual compaction pressure can be estimated. It is expected that the injection pressure will decay slightly during rest period as water drains out of the sample. In stage five the back volume that is connected to the injection system is kept constant and the pressure measured. The cell is then elevated to push against the model pile so that the load cell can measure the new pile penetration resistance.

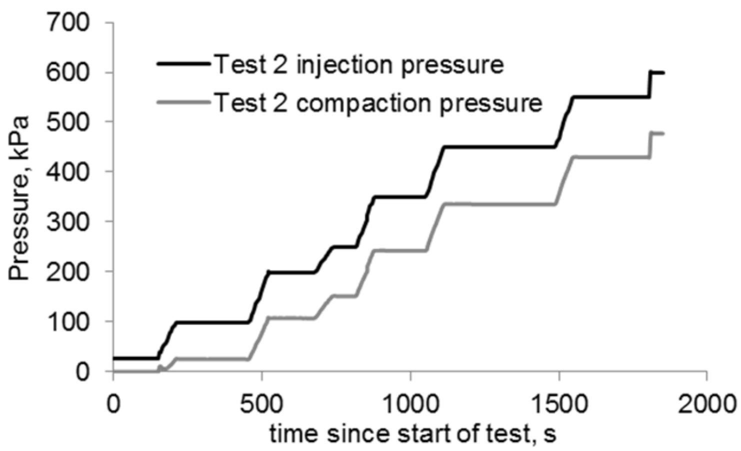

Figure 4. Compaction pressure estimated using injection pressure calibration curve in Figure 3b

3.1 Measurement of compaction rate

Compaction of the sample is expected to occur during driven pile penetration and also during compaction grouting. To estimate the compaction effect, it is assumed that the soil is completely saturated after the Skempton's B value check during the sample saturation stage reached 0.95. In a fully saturated soil, the voids between soil particles are completely filled with water and thus the volume of voids in the sample, V_v equals to the volume of water in the sample, V_w. Therefore the void ratio, e of the sample can be defined as equation 1 where V_s is the volume of soil particles.

$$e = \frac{V_v}{V_s} = \frac{V_w}{V_s} \tag{1}$$

During all stages of the experiment, the volume of soil particles remains the same. Then, the volume of water drained out is a measure of the change in void ratio.

$$\Delta e = e_0 - e = \frac{V_{w0}}{V_s} - \frac{V_w}{V_s} = \frac{(V_{w0} - V_w)}{V_s} = \frac{\Delta V_w}{V_s} \tag{2}$$

In granular soil, the density of the sand can be compared to its densest condition. This is usually done by calculating the sand's relative density, D_r from known void ratio e, the soil's loosest void ratio, e_{max} and soil's densest void ratio, $e_{min.}$

$$D_r = \frac{(e_{max} - e)}{(e_{max} - e_{min})} \tag{3}$$

$$D_r = \frac{(e_{max} - e_0)}{(e_{max} - e_{min})} - \frac{\Delta e}{(e_{max} - e_{min})} = \frac{(e_{max} - e_0)}{(e_{max} - e_{min})} - \eta \tag{4}$$

Rearranging equation 2 into equation 3 produces equation 4. The second portion of the equation 4 will be used to measure the compaction efficiency, η as defined by Wang et al (2010)

$$\eta = \frac{\Delta e}{(e_{max} - e_{min})} \tag{5}$$

4 EXPERIMENTAL RESULTS AND DISCUSSION

To demonstrate the use of this apparatus, we shall explore the results from test number 2 with initial dry density of 1499.5 kg per cubic metre. Results from Test 2 during the initial penetration stage demonstrated the ability of the test to show the effect of repeat pile testing on the pile penetration result. The compaction grouting stage is also demonstrated in Test 2. The change in compaction efficiency due to injection pressure increase is demonstrated here. The last penetration resistance before compaction grouting in Test 2 are then compared to the post-compaction grouting pile test results. The experiment completes all five stages and the effect of compaction grouting on driven pile capacity is demonstrated.

4.1 Penetration stage

The pile penetration stage is conducted in three consequent displacements of about 70 to 85mm (Figure 5). The penetration resistance is measured by the load cell. In driven pile terminology, the penetration resistance is expected to develop until it reaches the soil's ultimate bearing capacity. The model pile has smooth stainless steel wall and it is expected that the contribution of the skin friction is minimal. Only the final penetration resistance is of interest to the traditional engineers since it is equivalent to the bearing capacity force, F. According to Craig (1998), the bearing capacity pressure, q_f can be calculated from the overburden pressure, σ_0 and friction angle,ϕ' using equation 6.

$$q_f = \sigma_0'.\frac{1+\sin\phi}{1-\sin\phi}.e^{\pi.\tan\phi} \quad \textbf{(6)}$$

$$F = q_f A \quad \textbf{(7)}$$

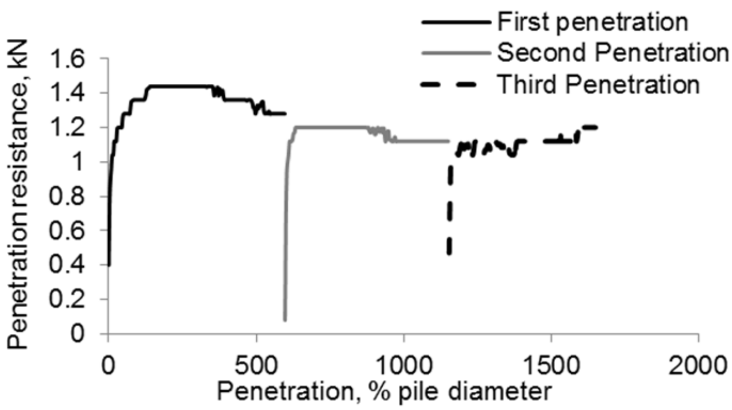

Figure 5. Pile penetration of loose Stockton Beach sand.

The first penetration depth took about 150% pile diameter to reach peak resistance of 1.4kN as the loose soil takes large displacement to densify at the tip of the model pile. During the second and third penetration the peak value of 1kN is reached after only 40 to 50% additional penetration since densification at piling stage has already occurred earlier. The result shows that driven pile needs to experience large distance of penetration in order to gain peak strength in loose soil. Such large deformation is disadvantageous for structures that could not tolerate large ground differential settlements. In addition to that, the operative driven pile capacity is usually lower after further penetration. It is of the interest of this experiment whether the peak resistance of 1.4kN can be regained by compaction grouting.

The value of measured force, F (Equation 7) is taken to be 1kN based on the final penetration resistance in Figure 5. To verify this value, equation 6 is used with peak ϕ' for loose to medium dense Stockton Beach sand between 33 and 40 degrees (Ajalloeian, 1996). According to Kishida (1967), the value of ϕ' to be used for pile driven in loose sand is an average of ϕ' obtained in the lab and 40. F is then calculated using equation 7 and the model pile cross section area, A. The calculated force F is between 0.65kN and 1.04kN. The value of 1kN measured is close to the value of medium dense sand as expected when sand is compacted by the advancing pile.

4.2 Compaction grouting stage

Compaction grouting is conducted by injecting the latex membrane with pressurised water. This stage is conducted after pile penetration for Test 2. The actual pressure exerted by the grout mass on the soil has to be estimated by knowing volume change (Figure 6a) and correlating it with balloon calibration pressure (Figure 3b). Test 2 was completed to maximum injection pressure of 600kPa with relaxation periods of 70 to 360 seconds between each pressure increment of 50 to 100kPa.

Figure 6 shows the amount of compaction in test 2. Since the sample size is large in comparison to the pile and grout mass, the compaction improvement will be based on change in compaction efficiency at various compaction pressures. Using equation 5, it is found that the value of η increased by 3.8 times when the grout injection pressure is increased from 200kPa at time 750s to 600kPa at time 1800s. The zone affecting the bearing capacity of the pile is theoretically within 3 to 4 times the pile diameter. Improving compaction in this region is vital in improving the bearing capacity of driven piles.

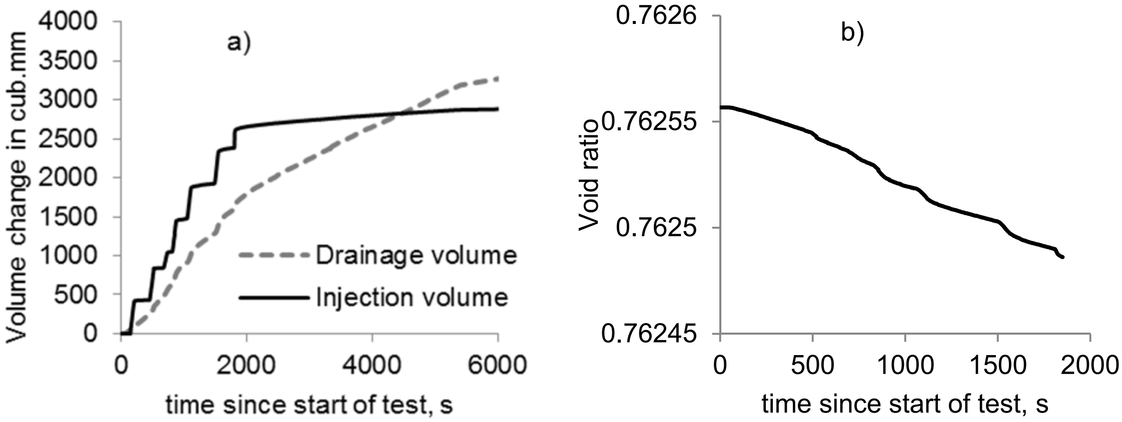

Figure 6. a) Volume changes during compaction grouting in Test 2; b) Void ratio changes in Test 2.

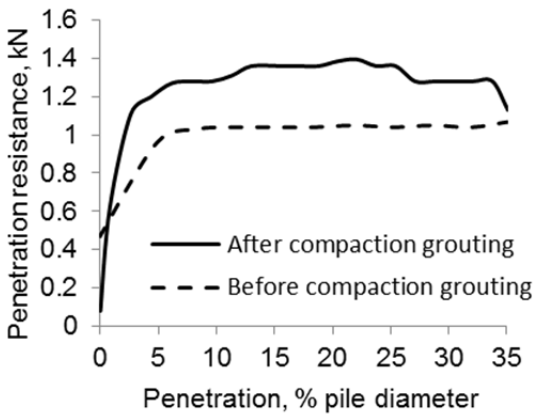

Figure 7. Pile penetration resistant before compaction grouting in Test 2 as compared to the same resistance after compaction grouting of 600kPa pressure in Test 2.

4.3 Second penetration stage

During this penetration test, the pile penetrated up to 20% of new pile diameter to reach peak resistance of 1.4kN in Figure 7. This is higher than 1kN pile capacity observed at the end of the third penetration test before compaction grouting in Test 2. It is also interesting to note that the pile penetration resistance has a peak of 1.4kN similar to the value of 1.4kN observed in Figure 5. This is a regain in strength of 0.4kN and an improvement of pile capacity by about 40%. A stiffer response is also observed in Figure 7. The penetration resistance has exceeded 1kN at less than 3% pile diameter penetration. This is better than 6% pile diameter penetration needed to reach 1kN penetration resistance for pile without compaction grouting.

5 CONCLUSION

To study the effect of compaction grouting on the improvement of pile capacity driven into Stockton Beach sand, a large triaxial apparatus was modified and used. The grouting technique used is unique in the sense that it is being injected radially into the soil rather than through the bottom of the pile tip.

It has been shown that compaction grouting could compact loose sand and reduce the void ratio of the soil region supporting the model pile. In doing so the density of the soil surrounding the compaction grout has increased and its operational strength has risen. It is also demonstrated that compaction improves by increasing the injection pressure.

It is evident that the compaction grout has created a larger bearing surface area at the bottom of the pile. The combined effect of increase in soil density and increase in pile bearing area contributed to the significant increase in pile bearing capacity. A 40% improvement in pile bearing capacity is expected and to mobilise the pile capacity, lesser settlement percentage is needed.

This experimental result demonstrated the successful use of the new apparatus in investigation of the improvement of driven pile capacity by compaction grouting. By combining driven pile and compaction grouting, the technology provides more flexibility for handling future foundation problems in difficult soils.

6 ACKNOWLEDGEMENTS

The work described in this paper is supported by ARC Discovery Project Grant DP1400509 and ARC CoE Early Career Award Grant CE110001009, for which the authors are very grateful. We also would like to acknowledge the contribution of the University of Newcastle civil engineering laboratory staff especially Mr. Lachlan Bates, Mr. Ross Gibson, Mr. Mick Goodwin and Mr. Javad Shemshad.

7 REFERENCES

Ajalloeian, R. (1996). An Experimental Study of Finite Pressuremeter Length Effects in Sand. Degree of Doctor of Philosophy, University of Newcastle.

Au, S., K. Soga and A. Yeung (2006). "A New Laboratory Apparatus for Grout Injection Studies." Geotechnical Testing Journal **29**(2): 7.

Bruce, D. (2005). "Glossary of Grouting Terminology." Journal of Geotechnical and Geoenvironmental Engineering **131**(12): 1534-1542.

Craig, R. F. (1998). Soil Mechanics, E & FN SPON, Routledge.

Fang, K., Z. Zhang, Q. Zhang and X. Liu (2013) "Prestressing effect evaluation for a grouted shaft: a case study." Proceedings of the ICE - Geotechnical Engineering 1-9.

Kishida, H. (1967). "Ultimate bearing capacity of piles driven into loose sand." Japanese Geotechnical Society 7(3): 10.

Mutman, U. and A. Kavak (2012). An in situ low-pressure grouting application. ICE-Geotechnical Engineering.

Pooranampillai, S., S. Elfass, W. Vanderpool and G. Norris (2010). "Large Scale Laboratory Testing of Low Mobility Compaction Grouts for Drilled Shaft Tips." Geotechnical Testing Journal **33**(5): 13.

Wang, S. Y., D. H. Chan, K. C. Lam and S. K. A. Au (2010). "Numerical and experimental studies of pressure-controlled cavity expansion in completely decomposed granite soils of Hong Kong." Computers and Geotechnics **37**(7–8): 977-990.