Proceedings of the 21st ICSMGE, Vienna, Austria, 14 – 19 June 2026. Pistrol, Adam & Schweiger (eds.)
Published by: ÖGG, Austrian Society for Geomechanics, Salzburg, Austria, ISBN 978-3-9503898-4-5
https://doi.org/10.53243/ICSMGE2026-1273

Field Testing of Expanding Resin Injection for Ground Improvement in Sands

Kyle Rollins, David Blake
Brigham Young University, Provo, Utah, USA, rollinsk@byu.edu, davidb@igesinc.com

Fredrick Wentz
Wentz-Pacific Ltd., Napier, New Zealand, rwentz@wp-geo.co.nz

Theo Hnat
Mainmark Ground Engineering, Ltd., Christchurch, New Zealand, thnat@mainmark.com

Sara Amoroso
University of Chieti-Pescara, Pescara, Italy; Istituto Nazionale di Geofisica e Vulcanologia, L'Aquila, Italy, sara.amoroso@unich.it

ABSTRACT: Expanding polyurethane resin injection is a ground improvement method for densifying loose sand susceptible to liquefaction. In contrast to many other improvement methods, resin injection can be employed without damage to existing infrastructure such as buildings, pipelines, and roadways. To investigate the potential of resin injection treatment to mitigate liquefaction and resulting settlement, two full-scale blast liquefaction tests were performed in Christchurch, New Zealand. One blast test was conducted within an 8-m diameter improved panel (IP) area with 50 injection points on a triangular grid, with injections occurring at 1.2-m on centers in loose to medium dense silty sands and sands. A similar blast test was performed on a companion untreated natural panel (NP). Improvement was evaluated using cone penetration tests (CPT), dilatometer tests (DPT), and shear wave velocity (V_s) testing before and after treatment. Treatment increased cone resistance by 68%, relative density by 32%, V_s by 25%, and DMT modulus by 134%. The average at-rest earth pressure coefficient, (Ko) also increased by 38%. The blast tests were severe enough that liquefaction was induced in both the IP and NP areas. Nevertheless, settlement from volumetric-strain during reconsolidation within the treated zone from 1-6 m depth was reduced by about 50% in comparison with the NP. Measured reconsolidation settlements were within the range of settlements computed using the CPT-based approach proposed by Zhang et al (2002) using CPT tests. The results of the study indicate that resin injection is a viable method of ground improvement to reduce liquefaction-induced settlements.

KEYWORDS: Expanding Resin Injection, Ground Improvement, Liquefaction Mitigation, Controlled blast Testing, In Situ Testing

1 INTRODUCTION

Earthquake-induced liquefaction has been a topic of great interest in the geotechnical community for many years because of the damage and economic losses it has inflicted in many earthquakes. Improving methods for predicting liquefaction has been at the forefront of much research. Likewise, mitigation of liquefaction potential has also become extremely prevalent and includes a wide range of methods. However, many mitigation methods cause ground surface settlement or other disruption to existing infrastructure. In contrast, expanding polyurethane resin injection is a ground improvement strategy well suited for sites with existing utilities or buildings because it does not induce settlement.

Resin injection has previously been used to level foundations of buildings that had settled over time (Erdemgil et al. 2007). In recent years, however, contractors have discovered that expanding polyurethane resin can densify sand in-situ, similar to compaction grouting, and reduce the potential for liquefaction (Traylen et al. 2018). In addition, the improvement equipment is relatively unobtrusive, lightweight, and can be used within existing buildings where headroom is limited. These are all important factors when considering what method of ground improvement to use for a project, as each method has its own limitations. However, polyurethane resin has typically been tested and vetted using in-situ test methods such as cone penetration testing (CPT), dilatometer testing (DMT), plate load testing (PLT) and direct push cross-hole testing (CH). At present there are no case histories documenting performance during earthquake shaking.

In 2013, the New Zealand Earthquake Commission (EQC) conducted a series of ground improvement trials to evaluate the technical viability and efficacy of multiple forms of shallow ground improvement methods. The methods that were tested focused on mitigating liquefaction damage to residential construction by forming a relatively stiff crust in the upper 3 to 4 m below the surface overlying liquefiable soils at depth (NZ EQC, 2015) although full treatment of the potentially liquefiable soils would clearly be desirable, it would not be economically viable for low-rise structures. This strategy allows liquefaction to occur at depth but relies on the stiffness of the surface crust to minimize differential settlement of structures at the surface as well as protrusions of sand ejecta. Sand ejecta was a major source of damage to residential structures in the Christchurch earthquake sequence (CES) (Quigley et al. 2013; Villemure et al. 2012). To verify the effectiveness of the tested ground improvement techniques in situ tests and geophysical measurements were also performed at these trial sites (e.g Amoroso et al. 2018).

In connection with this EQC trial, resin injection treatment was evaluated at three sites in the heavy damage (red) zone in Christchurch. Although soil improvement was produced by the treatment at all of these sites, blast testing was not performed making it difficult to compare the effectiveness of the resin injection with that of the other strategies.

This paper describes the soil improvement produced using polyurethane resin injection at one of the three liquefiable sand site in Christchurch, New Zealand. Improvement produced by resin injection in a liquefiable soil profile is evaluated by in-situ tests (i.e. CPT, DMT, and Cross-hole V_s) as well as controlled blasting. Controlled blasting can evaluate improvement in limiting excess pore pressure generation and the settlement produced by volumetric strain during reconsolidation. These tests, conducted at full-scale and in natural ground conditions, are highly valuable in understanding the field performance of treated sands during an earthquake (Ashford et al. 2004, Rollins et al. 2021, Amoroso et al. 2020).

2 SITE CHARACTERIZATION BEFORE TREATMENT

The test site was located at Breezes Road about 100 m east of the Avon River in Christchurch, New Zealand. Prior to the devastating Christchurch Earthquake Sequence (CES) of 2010 – 2011, this area was a densely populated residential neighborhood. Significant liquefaction-induced damage occurred during the sequence causing all of the residents to abandon their homes due to the devastating effects of the earthquakes (Cubrinovski and Robinson 2016) and the classification of the area as a 'red zone' by the government (Wilson 2015). By the end of 2015, all the homes in the red zone had been demolished.

Site characterization was performed using cone penetration tests with pore pressure measurements (CPTu or piezocone test), geophysical tests, and seismic dilatometer testing (SDMT). Data from CPTu and SDMT were obtained in 2016 before and after resin injection ground improvement. As shown in Fig. 1, the idealized soil profile based on these tests consists of silty sand (SM) with occasional silt bands to 2.5-3.0 m below ground level. Underlaying the SM layer is a fine to coarse sand (SP) which extends to 15 m (maximum depth of CPTu and SDMT soundings). Soil profiles for the NP and the IP are generally the same, with small variations in the location of the silt bands. The water table was 1.1 m below the ground surface. The silty sand in the upper layer of the pre-improved IP had a fines content of approximately 40%, while the deeper, cleaner sand layer had fines contents of only 1 to 3%. The measured tip resistances were between 2 and 4 MPa in the SM layer, but were typically 5 to 10 MPa in the SP layer. Relative density estimated using Robertson and Cabal (2022) indicates D_r of about 35-45% in the SM layer and about 50-65% in the SP before treatment. Cross-hole shear wave velocities averaged 125 – 140 m/s in the upper 3 m, and 140 – 175 m/s from 3 to 8 m (Traylen et al. 2018).

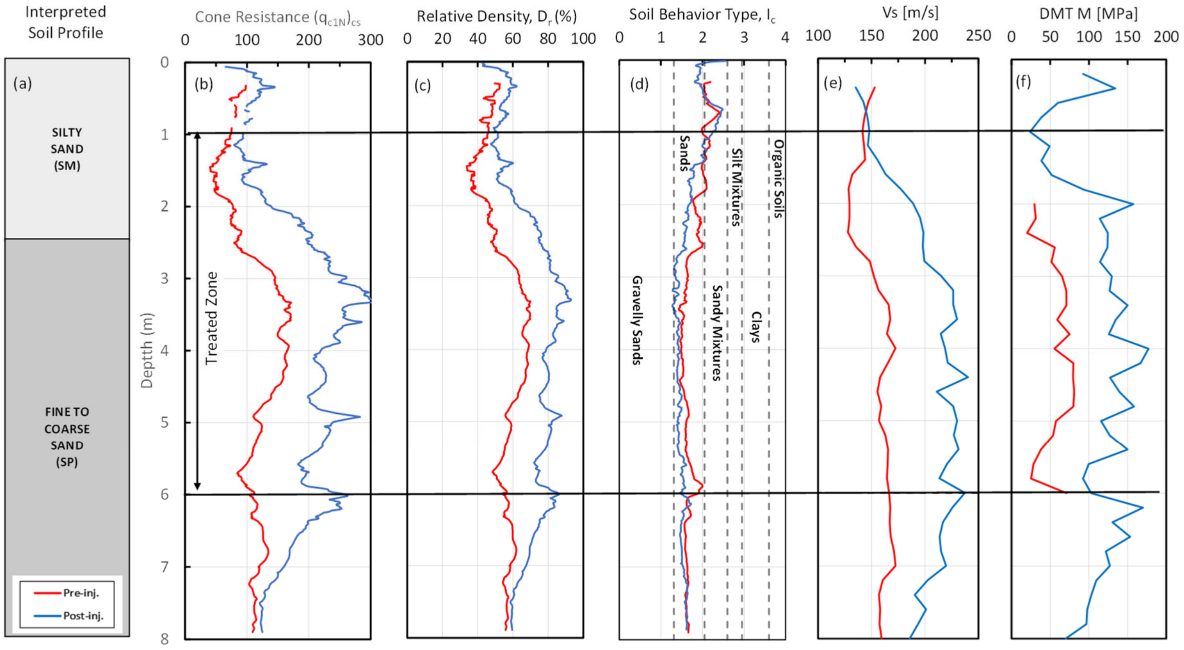

Figure 1. Summary of (a) soil profile, (b) CPT q_{c1Ncs}, (c) D_r, (d) I_c, (e) DMT M, and (f) V_s in improved panel before and after resin injection treatment.

3 RESIN INJECTION PROCESS

As shown in Fig. 2, the resin injection was performed using 50 grout injection tubes (16 mm diameter) laid out in a triangular pattern with a center-to-center spacing of 1.2 m. This created an improved volume from 1 to 6 m in depth and roughly 8 m in diameter. The resin injection process consisted of a hybrid method between the top-down and bottom-up processes. Initially, a capping layer was produced within a depth of 1 to 1.5 m, then the injection tube was pushed to 6 m and withdrawn at 0.5 m increments following the bottom-up method. A set volume of resin was injected at the base of the tube at each half-meter increment based on the outcomes of the pre-production test and the contractor's previous experience. Prior to injection, a surcharge pressure of about 14 kPa was applied to simulate a two-story residential structure. Average heave in the treatment zone was about 37 mm during the treatment. Fig. 2 also shows the location of various in-situ tests performed pre- and post-injection to verify the effectiveness of the improvement process.

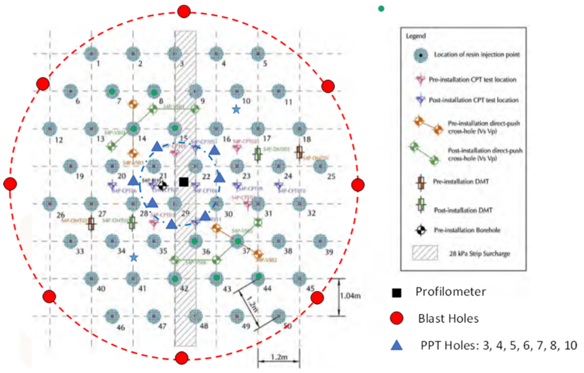

Figure 2. Layout of resin injection points. in-situ test locations, blast hole locations.

Fig. 1 also provides the in-situ test results from the CPT, DMT, and V_s testing at the improved panel after treatment in comparison with the untreated soil. The resin injection program significantly improved corrected cone tip resistance for clean sand (q_{c1Ncs}), relative density (D_r) shear wave velocity (V_s), and DMT constrained modulus (M). Subsequent CPT, DMT, and Vs testing three years after treatment showed the improvement had persisted. Table 1 provides a summary of the average improvement of the various soil properties within the IP resulting from treatment.

Table 1 Summary of increase in soil properties due to resin injection treatment

Soil Property	Percent Increase in Soil Parameter
q_{c1Ncs}	68%
D_r	32%
V_s	25%
M	134%

4 BLAST TESTING

4.1 *Blasting Process*

Two blast-induced liquefaction tests were conducted. One blast was carried out around a natural panel without any treatment while a second blast was carried out around the improved panel previously treated in September of 2016. For each blast test, a total of sixteen separate charges were placed in eight blast holes located around the periphery of a circle with an approximate diameter of 10 m (see Figs. 2 and 3). Each blast hole (10 m deep) was cased with a 75 mm diameter PVC pipe. Two decks of explosives were placed in each pipe: one 1.2 kg charge centered at a depth of 4 m and one 2.4 kg charge centered at a depth of 8.5 m. Thus, the total weight of the charges for each blast test was 28.8 kg. Gravel stemming was placed between the explosive charges and above the upper deck to the surface to help direct the energy in the horizontal direction rather than vertically up the blast hole. In each blast test, the lower deck of explosives was detonated sequentially with a 600-millisecond delay alternating from opposite sides of the panel circle to simulate the shearing induced during an earthquake. Afterwards, the upper deck of explosives was detonated with the same sequence.

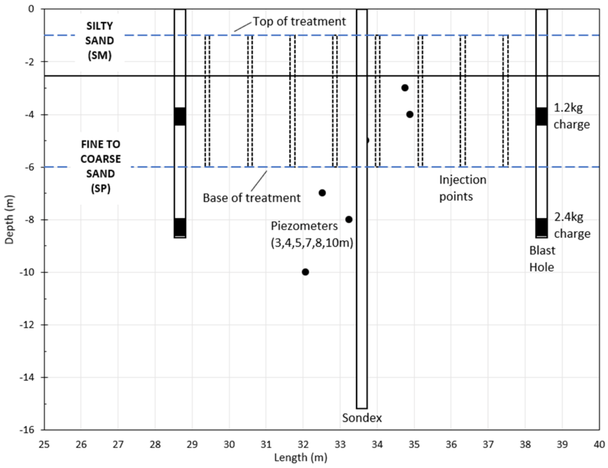

Figure 3. Cross-section showing treatment area, blast charge layout, and piezometers and Sondex pipe locations.

4.2 *Instrumentation*

Instrumentation consisted of piezometers, surface settlement stakes, and a Sondex settlement profilometer as shown in Fig. 3. The piezometers were located at depths of 3, 4, 5, 7, 8, and 10 m below the ground surface and recorded the generation and dissipation of pore pressure with time. The settlement at each surface survey stake measured with a high resolution autolevel provided liquefaction-induced settlement at the ground surface. The Sondex profilometer measured liquefaction-induced settlement within the treatment zone.

4.3 *Blast Test Results*

The maximum excess pore pressure ratio (R_u) versus depth in the natural and improved panels produced by the two separate blast tests are presented in Fig. 4. Ru is computed with the equation

$$R_u = \Delta u/\sigma'_{vo} \qquad (1)$$

where Δu is the excess pore pressure and σ'_{vo} is the initial vertical effective stress prior to the blast. Although, the resin injection treatment clearly increased the CPT tip resistance in the improved panel, as shown in Fig. 1, the blast energy was sufficient to produce excess R_u values that were between 0.9 and 1.0 in both panels indicating liquefaction.

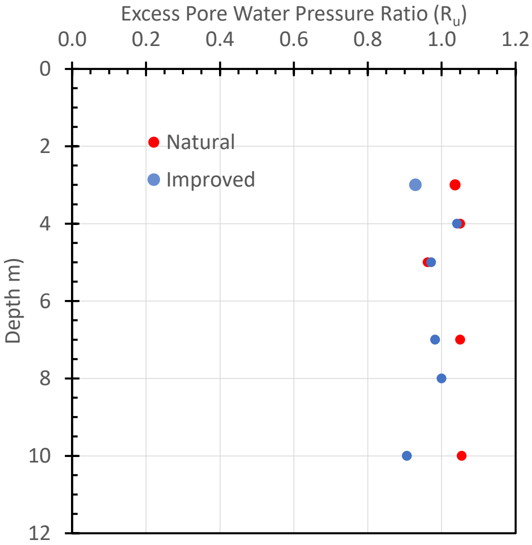

Figure 4. Maximum excess pore pressure ratio versus depth in the natural and improved panels produced by the two blast tests.

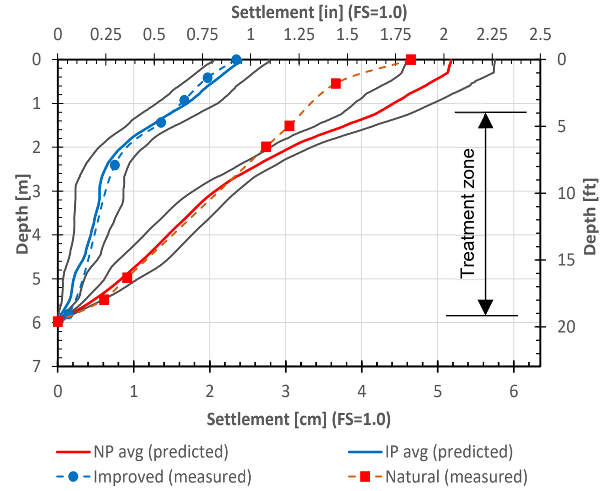

Figure 5. Measured liquefaction-induced settlement versus depth curves along with settlement computed using the Zhang et al. (2002) approach.

The settlement versus depth profiles measured within the top 6 m (treated zone from 1 to 6 m) of the soil profile for the natural and improved panels is shown in Fig. 5. Settlement has been zeroed at a depth of 6 m to facilitate comparisons. Maximum settlement in the treated zone was reduced from 4.6 cm in the improved panel to 2.3 cm in the natural panel; a reduction of 50%. About 30% of the settlement occurred within the upper one meter of the profile at both panels where there was no injection, but some blast-induced compaction occurred.

5 COMPUTATION OF LIQUEFACTION-INDUCED SETTLEMENT

Using the Zhang et al. (2002) CPT-based methodology, post-liquefaction settlements were calculated as part of this study using the equation

$$S = \sum_{i=1}^{j} \varepsilon_{vi} \Delta z_i \qquad (2)$$

where ε_{vi} is the post-liquefaction volumetric strain for a soil sublayer i, Δz_i is the thickness of sublayer i, and j is the total number of soil sublayers. The computed liquefaction-induced ground settlement versus depth at the NP and IP are also plotted in Fig. 5. The strains in the NP were computed using with the average CPT profile soundings obtained prior to the polyurethane resin treatment in 2016 and a CPT conducted in November 2019. The average computed settlement in the top 6 m of the NP is 52 mm. In the IP, 24 mm of settlement is predicted in the upper 6 m, based on the average CPT profile from one month after the ground improvement was carried out along with a sounding conducted within the IP in November of 2019. For both panels, the upper and lower bound settlement versus depth curves were computed using the mean ±1 standard deviation CPT values. A review of the measured and computed curves in Fig. 5 indicates that the agreement is relatively good. The measured settlement typically falls within computed mean ± one standard deviation settlement bounds in Fig. 5.

6 OBSERVATIONS AND CONCLUSIONS

Based on the results from the resin injection treatment and the blast-induced liquefaction testing the following observations and conclusions are drawn:

1. Resin injection grouting produced significant increases in the q_{c1Ncs}, D_r, V_s, and M in the treated sand. These improvements were confirmed by tests conducted three years after the treatment in 2016 and confirm the persistence of the improvement over time.
2. The increase in q_{c1Ncs} and D_r produced a significant increase in liquefaction resistance and a reduction in liquefaction-induced settlement.
3. Despite the increase in liquefaction resistance, the energy of the blast testing was sufficient to produce liquefaction in both the natural and improved panels. Nevertheless, liquefaction-induced settlement in the treated layer of the IP was about 50% of the settlement recorded over the same depths of the NP.
4. Using the Zhang et al (2002) CPT-based methodology (FS=1.0) for predicting liquefaction-induced ground settlements, good correlation was found between observed and calculated settlements for the depths of interest in both the IP and NP. Measured settlement was typically within the computed mean ± one settlement bounds from the CPT soundings in each panel.

7 ACKNOWLEDGEMENTS

Funding for the blast testing in this study was provided by the National Science Foundation Grant CMMI-1926245. Mainmark Ground Engineering designed and performed the resin injection. This support is gratefully acknowledged; however, any opinions, findings, or conclusions expressed in this paper are those of the authors and do not necessarily reflect the views of the National Science Foundation.

8 REFERENCES

Ashford, S. A., Rollins, K. M., and Lane, J. D. (2004). "Blast-induced liquefaction for full-scale foundation testing." *Journal of Geotechnical and Geoenvironmental Engineering*, 130(8), 798-806.

Amoroso, S., K. M. Rollins, P. Monaco, M. Holtrigter, and A. Thorp. 2018. “Monitoring ground improvement using the seismic dilatometer in Christchurch, New Zealand.” Geotech. Test. J. 41 (5): 946–966. https://doi.org/10.1520/GTJ20170376.

Amoroso, S., Rollins, K. M., Andersen, P., Gottardi, G., Tonni, L.,

García Martínez, M. F., Wissmann, K., Minarelli, L., Comina, C., Fontana, D., De Martini, P. M., Monaco, P., Pesci, A., Sapia, V., Vassallo, M., Anzidei, M., Carpena, A., Cinti, F., Civico, R., Coco, I., Conforti, D., Doumaz, F., Giannattasio, F., Di Giulio, G., Foti, S., Loddo, F., Lugli, S., Manuel, M. R., Marchetti, D., Mariotti, M., Materni, V., Metcalfe, B., Milana, G., Pantosti, D., Pesce, A., Salocchi, A. C., Smedile, A., Stefani, M., Tarabusi, G., and Teza, G. (2020). "Blast-induced liquefaction in silty sands for full-scale testing of ground improvement methods: Insights from a multidisciplinary study." *Engineering Geology*, 265, 105437.

Cubrinovski, M., and Robinson, K. (2016). "Lateral Spreading: Evidence and interpretation from the 2010–2011 Christchurch earthquakes." *Soil Dynamics and Earthquake Engineering*, 91, 187-201.6

Erdemgil, M., Sağlam, S., and Bakır, B. "Utilization of Highly Expansive Polymer Injection to Mitigate Seismic Foundation Failure for Existing Structures." *Proc., Proc. 8th Pacific Conference on Earthquake Engineering.*

NZ EQC (2015). "Residential Ground Improvement: Findings from trials to manage liquefaction vulnerability."

Quigley, M. C., Bastin, S., and Bradley, B. A. (2013). "Recurrent liquefaction in Christchurch, New Zealand, during the Canterbury earthquake sequence." *Geology*, 41(4), 419-422.

Robertson, P.K. and Cabal, K. (2022). "Guide to CPT Testing-7th Edition“ Gregg Drilling, Inc., 156 p.

Rollins, K. M., Amoroso, S., Andersen, P., Tonni, L., and Wissmann, K. (2021). "Liquefaction Mitigation of Silty Sands Using Rammed Aggregate Piers Based on Blast-Induced Liquefaction Testing." *Journal of Geotechnical and Geoenvironmental Engineering*, 147(9), 04021085.

Traylen, N. J., Wentz, F. J., Van Ballegooy, S., Wotherspoon, L. M., Hnat, T., and Deller, R. (2018). "A Study into Resin Injection as a Ground Improvement Technique for Seismic Liquefaction Mitigation." *Geotechnical Earthquake Engineering and Soil Dynamics V: Liquefaction*

Villemure, M., Wilson, T., Bristow, D., Gallagher, M., Giovinazzi, S., and Brown, C. (2012). "Liquefaction ejecta clean-up in Christchurch during the 2010-2011 earthquake sequence."

Wilson, J. (2015). "'Canterbury places - East Christchurch', Te Ara - the Encyclopedia of New Zealand." <http://www.TeAra.govt.nz/en/zoomify/46379/eastern-suburbs-red-zone>. (January 27, 2021).

Zhang, G., Robertson, P., and Brachman, R. W. (2002). "Estimating liquefaction-induced ground settlements from CPT for level ground." *Canadian Geotechnical Journal*, 39(5), 1168-1180.