Performance of Resin Injection Ground Improvement in Silty Sand Based on Blast-Induced Liquefaction Testing in Christchurch, New Zealand

David Harold Blake
Brigham Young University

Follow this and additional works at: https://scholarsarchive.byu.edu/etd

Part of the Engineering Commons

BYU ScholarsArchive Citation
Blake, David Harold, "Performance of Resin Injection Ground Improvement in Silty Sand Based on Blast-Induced Liquefaction Testing in Christchurch, New Zealand" (2022). Theses and Dissertations. 9458. https://scholarsarchive.byu.edu/etd/9458

This Thesis is brought to you for free and open access by BYU ScholarsArchive. It has been accepted for inclusion in Theses and Dissertations by an authorized administrator of BYU ScholarsArchive. For more information, please contact ellen_amatangelo@byu.edu.
Performance of Resin Injection Ground Improvement in Silty Sand Based on Blast-Induced Liquefaction Testing in Christchurch, New Zealand

David Harold Blake

A thesis submitted to the faculty of Brigham Young University in partial fulfillment of the requirements for the degree of

Master of Science

Kyle M. Rollins, Chair
Norman L. Jones
Gus P. Williams

Department of Civil and Construction Engineering Brigham Young University

Copyright © 2022 David Harold Blake
All Rights Reserved
ABSTRACT

Performance of Resin Injection Ground Improvement in Silty Sand Based on Blast-Induced Liquefaction Testing in Christchurch, New Zealand

David Harold Blake
Department of Civil and Construction Engineering, BYU
Master of Science

Polyurethane resin injection is a treatment being considered as a replacement for traditional methods of ground improvement. It has been used to re-level foundations and concrete slabs that have settled over time. Additional claimed benefits of the treatment have been noted recently, including improved factors of safety against soil liquefaction and reduced earthquake-induced settlements. To investigate the capability of the polyurethane resin injection treatment to mitigate liquefaction, two full-scale blast liquefaction tests were performed; one test was conducted in an improved panel (IP), an 8 m circular area treated with the polyurethane resin in a 1.2 m triangular grid from a depth of 1 to 6 m, and another test in an untreated 8 m circular area, the natural panel (NP).

Each blast test was severe enough to produce liquefaction ($r_u \approx 1.0$) in the respective panel, with blast-induced settlements in the range of 70 to 80 mm. Despite similar levels of ground-surface settlement in the IP and NP, settlement within the top 6 m of the IP was about half that of the NP. A CPT-based predicted settlement for each panel was employed using the Zhang et al. (2002) methodology. Good correlation was found between the observed settlements and predicted settlements in both panels. Differential settlements across the panels were calculated based on ground-based lidar surveys, with a reduction of 42 to 49% between the IP and NP. The measured total and differential settlements following resin injection were at the bottom of the range observed in blast tests on a variety of shallow ground improvement methods conducted by the New Zealand Earthquake Commission in 2013. The persistence of the polyurethane resin injection ground improvement three years following its installation was indicated by the lasting increase of fundamental in situ test parameters. The results of the study indicate that resin injection is a viable method of ground improvement to reduce liquefaction-induced settlements by creating a stiffer surficial crust.

Keywords: resin, ground improvement, liquefaction, liquefaction mitigation, blast-induced liquefaction
ACKNOWLEDGEMENTS

Funding for this study was provided by the National Science Foundation Grant CMMI-1926245. This support is gratefully acknowledged; however, any opinions, findings, and conclusions or recommendations expressed in this material are those of the author(s) and do not necessarily reflect the views of the National Science Foundation.

Funding for the ground-based lidar and survey data were provided, in part, by the National Science Foundation Rapid Award CMMI-2002382 (PI: Jonathan Hubler, Villanova). This work also utilized equipment from the NSF Natural Hazards Engineering Research Infrastructure: Post-Disaster, Rapid Response Research (RAPID) Facility (Award CMMI-1611820). Additional equipment and/or software were provided by Leica Geosystems and David Evans and Associates, Maptek I-Site and CloudCompare.

I wish to express my gratitude to Dr. Kyle M. Rollins for his patient guidance and mentorship throughout my time as a graduate student. His knowledge, kindness, and wit are unparalleled and his love for geotechnical engineering is truly inspiring.

I also express appreciation for my wife, Carlota, and daughter, Elizabeth, who have supported and encouraged me every step of the way throughout my graduate studies. Their love and support have carried me through the most difficult times and given me hope for a brighter future.
TABLE OF CONTENTS

LIST OF TABLES ......................................................................................................................... vi
LIST OF FIGURES ........................................................................................................................ vii

1 Introduction .............................................................................................................................. 1
  1.1 Research objectives .......................................................................................................... 2
  1.2 Scope of work ................................................................................................................... 2
  1.3 Outline of report ............................................................................................................... 4

2 Liquefaction .............................................................................................................................. 5
  2.1 Introduction to liquefaction .............................................................................................. 5
  2.2 Liquefaction-induced settlement ...................................................................................... 8
  2.3 Liquefaction evaluation by CPT .................................................................................... 13
  2.4 Summary ........................................................................................................................ 21

3 Liquefaction mitigation and resin injection treatment ............................................................ 22
  3.1 Existing mitigation techniques ....................................................................................... 22
  3.2 Resin injection as a method of ground improvement ..................................................... 26
    3.2.1 Construction methodology ...................................................................................... 27
    3.2.2 Current state of resin injection ground improvement research ................................ 29

4 Preliminary investigations and site characterization .............................................................. 35
  4.1 Geotechnical conditions ................................................................................................. 35
  4.2 Liquefaction assessment ................................................................................................. 43

5 Ground improvement procedure ............................................................................................. 45
  5.1 Injection of resin improvement ...................................................................................... 45
    5.1.1 Ground heave .......................................................................................................... 47
  5.2 Post-injection ground improvement evaluation ............................................................. 48
    5.2.1 CPT ......................................................................................................................... 49
    5.2.2 Shear wave velocity, compression wave velocity, and shear modulus .................... 52
    5.2.3 Dilatometer constrained modulus (M) and horizontal stress index (K_D) ............... 56
    5.2.4 In-situ at-rest earth pressure coefficient (K_o) ........................................................ 58
    5.2.5 Persistence of treatment .......................................................................................... 62

6 Site instrumentation and blast test procedure ......................................................................... 63
  6.1 Explosives setup and blast procedure ............................................................................. 63
LIST OF TABLES

Table 2.1: Relationship between post-liquefaction volumetric strain and \((q_{c1N})_{cs}\) for various factors of safety (Zhang et al. 2002)........................................................................................................13
Table 4.1: Site 3 IP testing schedule...........................................................................................................37
Table 5.1: Standard deviation and coefficient of variation of \(K_0\) for plot (a) in Figure 5.9 .............61
Table 5.2: Increases in soil parameters within the treated zone ..............................................................62
Table 6.1: Initial vertical effective stress readings at each pore pressure transducer used in the computation of \(r_u\) during blast activities .........................................................................................72
Table 6.2: Location of the four PVC surface settlement posts and their distances from the natural and improved panels........................................................................................................77
Table 7.1: Average settlement for individual concrete blocks and general kentledge area.............104
LIST OF FIGURES

Figure 2.1: Example of flow liquefaction failure at the Sheffield Dam, 1925 (University of California, Los Angeles). .................................................................6

Figure 2.2: Development of sand ejecta and flooding following a Mw 6.3 earthquake on February 22, 2011 (New Zealand Herald). .......................................................7

Figure 2.3: Chart for determining volumetric strains as functions of factor of safety (Ishihara and Yoshimine 1992). ..........................................................11

Figure 2.4: Relationship between post-liquefaction volumetric strain and equivalent clean sand normalized CPT tip resistance for different factors of safety (Zhang et al. 2002). ..................................................12

Figure 2.5: CPT-based soil behavior type chart proposed by Robertson (1990). ..........................................................16

Figure 2.6: Variations of stress reduction coefficient with depth and earthquake magnitude (Idriss 1999). .................................................................18

Figure 2.7: Example plot of CRR and CSR versus depth to determine where liquefaction may occur. .................................................................20

Figure 2.8: Typical relationships between residual excess pore pressure ratio and FS_L (Marcuson III et al. 1990). ..........................................................21

Figure 3.1: (Left) Hand-exhumed resin veins; (Right) Hydro-exhumed resin veins (Traylen et al. 2017). .................................................................27

Figure 3.2: Comparison of (a) pre- and (b) posttreatment test results: horizontal section of the 3D resistivity model at a depth of 125 cm and Q_c contour lines by interpolation (Apuani et al. 2015). .................................................................32

Figure 4.1: Residential neighborhood in September 2010 (left) and the same area in November 2015 (right) following the CES. ............................................37

Figure 4.2: Comparisons of CPTu test results at the NP and the IP prior to resin injection with respect to (a) cone resistance, q_t, (b) sleeve friction, f_c, (c) porewater pressure, U_2, (d) soil behavior type, I_c, (e) ratio of constrained modulus to cone tip resistance, M/q_t, and (f) interpreted soil profile. ..................................................38

Figure 4.3: Comparisons of SDMT test results at the NP and the IP prior to resin injection with respect to (a) horizontal stress index, K_D, (b) soil material index, I_D, (c) constrained modulus, M, (d) fines content, FC, (e) shear wave velocity, v_s, and (f) interpreted soil profile. ..................................................39

Figure 4.4: A comparison of the (a) horizontal stress index and (b) constrained modulus between the NP and IP based on December 2019 SDMT testing. .............................................40

Figure 4.5: Grain size curve of soil at Breezes Road test site at a depth of 5 m ..........41
Figure 4.6: Soil behavior type plot consisting of CPTu data from a sounding in the IP prior to treatment.

Figure 4.7: (a) Cyclic stress ratio (CSR) and cyclic resistance ratio (CRR), (b) Factor of safety against liquefaction in the pre-treated IP.

Figure 5.1: Aerial photograph at the Breezes Road (Site 3) test site showing the setup for and installation of resin using the Mainmark Teretek™ system (Traylen et al. 2017).

Figure 5.2: Plan view of test area showing layout of resin injection points along with locations of CPT, DMT, and Vs holes (Traylen et al. 2017).

Figure 5.3: CPT cone resistance comparison of (a) before and after improvement, (b) pre-improvement and 2019 natural panel, and (c) post-improvement and 2019 improved panel.

Figure 5.4: Comparisons of CPTu test results at the IP before and after treatment with respect to (a) cone resistance, qc, (b) equivalent clean sand normalized cone tip resistance, (qc1N)cs, (c) soil behavior type, Ic, and (d) ratio of constrained modulus to cone tip resistance, M/qt, and (e) interpreted soil profile.

Figure 5.5: (a, b) Two compression wave (Vp) velocity profiles at the test site, (c) shear wave velocity (Vs) before and after treatment, and (d) Vs over time.

Figure 5.6: Normalized shear wave velocity vs. excess pore pressure ratio from blast liquefaction tests.

Figure 5.7: Normalized shear modulus vs. excess pore pressure ratio from blast liquefaction tests in comparison with theoretical curve proposed by Kramer and Greenfield (2017) and empirical data based on data from Kinney (2018).

Figure 5.8: Comparisons of average pre- and post-injection (2016) (a) horizontal stress index and (b) constrained modulus to December 2019 SDMT testing at the IP.

Figure 5.9: Comparison of K0 between the pre-improved IP and (a) post-improvement IP a month after treatment, (b) post-improvement IP three years after treatment, and (c) NP.

Figure 6.1: Capped PVC pipe where charges were lowered to the appropriate depths.

Figure 6.2: Aerial image of NP showing capped blast holes.

Figure 6.3: Blast hole configuration in the IP with respect to the injection points.

Figure 6.4: Diagram of typical pore pressure transducer configuration (Lusvardi 2020).

Figure 6.5: Plan view of pore pressure transducer layout (with depths) across the test site.

Figure 6.6: (Left) Profile view of profilometer; (Right) Picture of metal rings attached to corrugated pipe for settlement readings.

Figure 6.7: Oblique view of the four grey PVC surface settlement posts and the wooden surface settlement stakes across the test site (IP in the foreground).
Figure 6.8: Locations of the surface survey stakes and posts across the test site.

Figure 6.9: Locations of the blast holes, resin injection points, pore pressure transducers, accelerometers, and survey equipment for settlement measurements.

Figure 6.10: Typical cross section of the improved panel showing instrumentation and injection point layout.

Figure 7.1: Residual excess pore pressure ratio in the IP during blast 1 in the IP at 3, 4, 5, 7, 8, and 10 m depths. Average peak residual pore pressure ratio with depth during blast sequence (shown inset).

Figure 7.2: Residual excess pore pressure ratio in the NP during blast 1 in the IP at 3, 4, 5, 7, 8, and 10 m depths.

Figure 7.3: Comparison of peak excess pore pressure ratio, \( r_u \), measured during blast 1 in the IP and in the NP.

Figure 7.4: Comparison of peak excess pore pressure ratio, \( r_u \), measured during blast 2 in the NP and in the IP.

Figure 7.5: Residual excess pore pressure ratio in the NP during blast 2 in the NP at 3, 4, 5, 7, and 10 m depths. Average peak residual pore pressure ratio with depth during blast sequence (shown inset).

Figure 7.6: Residual excess pore pressure ratio in the IP during blast 2 in the NP at 3, 4, 5, 7, 8, and 10 m depths.

Figure 7.7: Comparison of peak excess pore pressure ratio, \( r_u \), measured during blast 1 in the IP and blast 2 in the NP.

Figure 7.8: Comparison of \( r_u \) calculated from PPT and SDMT in NP for blast 2.

Figure 7.9: Dissipating \( r_u \) values with depth between 10 and 240 seconds after initial detonation during blast 1 and blast 2.

Figure 7.10: Pore pressure generation in the improved panel at 3 and 7 m depths.

Figure 7.11: Comparison of pore pressure generation in the NP and IP at 3 m depth.

Figure 7.12: (Left) Ejecta observed at the IP after Blast 1; (Right) Ejecta observed at the NP after Blast 2.

Figure 7.13: (a) Measured ground settlement with time for the IP during blast 1 and (b) for the NP during blast 2. Normalized ground settlement and average \( r_u \) for (c) the IP in blast 1 and (d) the NP in blast 2.

Figure 7.14: Superimposed ground surface settlements of IP and NP based on measurements with the wooden survey stakes.

Figure 7.15: Settlement across the test site on a line between the center of the IP and NP.

Figure 7.16: Overall site settlement based on ground-based lidar data.
Figure 7.17: Blocked polygons in NP and IP overlaying lidar data.................................103

Figure 7.18: Average settlement of blocks in the IP and NP for corroborating differential settlement calculations.................................................................................................................................106

Figure 7.19: Differential settlement vs total settlement (Wentz et al. 2015)......................107

Figure 7.20: Tilt-removed differential settlement vs total settlement (Wentz et al. 2015)......108

Figure 7.21: Tilt-removed differential settlement vs differential settlement (Wentz et al. 2015).................................................................................................................................108

Figure 7.22: Comparison of profilometer settlement vs. depth profiles with predicted settlement using the Zhang et al. (2002) CPT-based method.................................................................110

Figure 7.23: Computed free-field liquefaction settlement with varying peak ground accelerations.................................................................................................................................111

Figure 7.24: An example accelerogram from AA1 during blast #2 recorded in the time domain in the X, Y, and Z directions.................................................................114

Figure 7.25: Time history of accelerations in the X, Y, and Z directions from Accelerometer 1 during the 5th 2.4 kg detonation of blast #2.................................................................115
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. Understanding the mechanisms that indicate the likelihood of liquefaction in the event of an earthquake are at the forefront of much research. 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. Polyurethane resin is a material which is being considered as a replacement for traditional methods of ground improvement for soils that are prone to liquefaction, especially for sites with existing utilities or superstructures because it does not induce settlement.

Resin injection has previously been used to level foundations of buildings that have settled over time. In recent years, however, contractors have discovered that resin injection may have additional benefits to the soil other than solely to re-level a foundation. Advantages to using resin injection include the ability to improve ground below existing structures through densification without inducing ground settlement, improved factor of safety against soil liquefaction, and reduced earthquake-induced settlement. In addition, the improvement equipment is relatively unobtrusive, lightweight, and can be used within 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 only 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 crosshole testing ($V_s$ and $V_p$). At present there are no case histories documenting performance during earthquake shaking. Although blast-induced liquefaction testing is highly valuable for understanding the effects the resin has on the soil’s properties during an earthquake.

1.1 Research objectives

The research objectives for this project were as follows:

1. Identify the persistence of the resin injection ground improvement process over time by means of in situ testing.

2. Add to the database defining the effect of surface crust thickness and stiffness on the settlement and distortion of structures on the crust.

3. Compare the efficacy of polyurethane resin injection as a method of shallow ground improvement to existing methods tested by the New Zealand Earthquake Commission.

4. Provide a direct comparison of excess pore water pressure and soil settlement in the panel treated with resin injection (improved panel, IP) versus the untreated panel (natural panel, NP).

1.2 Scope of work

To accomplish these objectives, two blast-induced liquefaction tests were conducted by researchers from Brigham Young University (BYU) at Site 3 (Breezes Road, Avondale) in Christchurch, New Zealand, where resin had previously been injected into the ground during the 2016 Resin Injection Ground Improvement Research Trials (RZT) (Traylen 2017).
Two circular panels, approximately 10 m in diameter, were laid out: the improved panel and the natural panel. The improved panel contained the resin injection ground improvement which was installed during the 2016 RZT. The natural panel was located to the southwest of the improved panel approximately 30 meters away center-to-center and was composed of natural undisturbed soils without any ground improvement.

Piezometers were installed in both panels at various depths to record the pore water pressures during each blast sequence. Downhole accelerometer arrays were installed around the center of the natural panel, as well as at a location midway between the center of the natural panel and the improved panel. These arrays were installed to gather ground acceleration information during each of the blast sequences. Additionally, a BlastMate (Instantel) was used to measure the peak particle surface velocity at each site during the blast for safety purposes.

Wooden survey stakes were installed in a straight line, from the center of the improved panel to the center of the natural panel, extending approximately 8 m past each panel. These stakes were used to create a ground surface profile to evaluate ground settlement before and after each blast sequence. An auto-level was used to measure relative heights of each survey stake, which was then standardized to a stable reference point located far from the blast sites. Profilometer tubes were installed near the center of each panel to define settlement versus depth profiles for each panel based on pre- and post-blast measurement. Survey rods were installed 1 m and 2 m from the center of each panel to measure ground surface settlement versus time using auto-level measurements during each blast sequence.

The target liquefiable layers at the test site corresponded to the depths at which the resin injection ground improvement was installed, with additional potential liquefaction occurring below the treated layer. This experiment will further investigate the influence of increased lateral
earth pressures, soil densification and increased surface crust stiffness from composite action on liquefaction resistance.

1.3 Outline of report

This thesis presents the details regarding the development of the case study, actions to execute the testing, and results and conclusions from the data obtained. The remainder of the thesis contains eight chapters.

Chapter 2 contains an overview of the current research pertaining to liquefaction and its effects on foundations. Chapter 3 describes common methods of ground improvement for liquefaction mitigation. This chapter also describes the process of resin injection installation. Chapter 4 reviews the preliminary investigations and analyses used for selecting a site to conduct the blast testing. Chapter 5 summarizes the site-specific installation details of the resin injection process and discusses the post-injection ground improvement evaluation. Chapter 6 describes the procedure for the blast testing as well as an overview of the instrumentation used as the test site. Chapter 7 explains the results from the blast test with respect to measured pore pressures, measured ground surface settlements, settlements with depth, ground-based lidar surveys, as well as recorded accelerograms from blasting. Chapter 8 is a summary of the completed work and presents conclusions and recommendations based on the findings of the tests.
2 LIQUEFACTION

2.1 Introduction to liquefaction

The term liquefaction has been used in conjunction with a variety of phenomena involving soil deformations of cohesionless soils under saturated, undrained conditions. Traditionally it is defined as the act or process of transforming any substance into a liquid. In cohesionless soils, the transformation is from a solid state to a liquefied state as a consequence of increased pore water pressure and reduced effective stress (Marcuson 1978). The generation of excess pore pressure under these conditions is an indicator for all liquefaction phenomena. While cohesionless soils are in a saturated state, the rapid loading caused by earthquakes or other strong ground motions creates an undrained condition, causing excess pore pressures to increase and therefore decreasing the effective stresses. When the generated pore pressure equals the confining pressure, the effective stress is zero and liquefaction is initiated (Studer and Kok 1980). As pore pressures dissipate, the liquefied sand reconsolidates into a denser state which produces volumetric strain or soil settlement.

Lateral displacement phenomena induced by liquefaction can be divided into two main groups: flow liquefaction and cyclic mobility (Kramer 1996).

Flow liquefaction is the phenomenon when static equilibrium is disrupted by static or dynamic loads in a soil with low residual strength. When this occurs, the soil is no longer able to withstand the static stresses that were acting on the soil before the disturbance occurred. This
results in practically zero residual strength, leading to large and rapid movements known as flow failures, such as those that occurred causing the failure of the Sheffield Dam near Santa Barbara California in 1925 (Seed et al. 1969) shown in Figure 2.1. The driving forces behind this kind of failure are the static shear stresses, which exceed the reduced shear strength and result in material instability (Andrade et al. 2013).

Cyclic mobility is the phenomenon when static shear stresses are less than the shear strength of the liquefied soil. Unlike deformations caused by flow liquefaction, deformations induced by cyclic mobility are developed incrementally because of the static and dynamic stresses that occur during an earthquake. This shear strength is sufficient to prevent flow failure, but insufficient to prevent lateral spread. These deformations, called lateral spreading, can occur on very gently sloped ground or almost flat ground (1 to 5% slope) adjacent to bodies of water. The combination of static and inertial shear stresses transiently exceeding the post-liquefaction strength allows for incremental down-slope movements to occur, the accumulation of which produce lateral spread displacements (Kramer 1996).

Figure 2.1: Example of flow liquefaction failure at the Sheffield Dam, 1925 (University of California, Los Angeles).
Liquefaction can also cause significant damage due to vertical displacements (or settlement) even under level-ground conditions when lateral displacement does not occur (Kramer 1996). This is caused by the upward flow of water that occurs when seismically- or blast-induced excess pore pressures dissipate. Rather than producing lateral soil movement, significant vertical settlement and consequent flooding occur. The upward flow of water carries sand ejecta to the surface forming sand boils or sand volcanos that are characteristic of level-ground liquefaction failure (Ishihara and Yoshimine 1992). During the 2010 – 2011 Canterbury Earthquake Sequence (CES) this type of liquefaction occurred and caused extensive damage, especially within residential areas (see Figure 2.2), estimated at $30 billion (NZD) to repair. Some 60,000 residential properties and buildings were affected by liquefaction and of those, about 8,000 were abandoned because of the excessive damage resulting from liquefaction and lateral spreading (Cubrinovski and Robinson 2016).

Figure 2.2: Development of sand ejecta and flooding following a M$_w$ 6.3 earthquake on February 22, 2011 (New Zealand Herald).
The effects of liquefaction are detrimental regardless of the type of liquefaction that occurs. Mitigating the results of liquefaction has been a topic of great interest over the past decades and many developments have been made in this field of study. Continuing research in this field leads to greater life and property safety in the event of a major earthquake.

2.2 Liquefaction-induced settlement

When soils possess the characteristics of liquefiable materials, certain factors should be considered during the design process. In the case study presented in this thesis, liquefaction was triggered through detonation of small explosive charges at a level ground site with a silty sand and sandy silt profile to determine the efficacy of a proprietary ground improvement technique which will be discussed in a later section. Liquefaction through blasting can induce settlements in both dry and saturated sands, though only the saturated case will be examined in this study (Dowding and Hryciw 1986). Previous studies indicate that the liquefaction-induced settlement from blasting is often similar to that expected from earthquake-induced liquefaction (Rollins et al. 2021).

During ground shaking, the soil matrix tends to contract due to induced shear stresses, causing an increase in pore water pressure. If these pressures equal the confining pressure, liquefaction occurs. In a liquefied state, the residual shear strength for loose to medium dense sand significantly decreases to a small percentage of its static resistance (Nagase and Ishihara 1988). Foundations overlying these liquefied layers may experience local bearing capacity failure resulting in extensive structural damage.

Once pore water pressures have dissipated following ground shaking, the soil particles rearrange into a denser configuration. This process, or volumetric strain, leads to ground subsidence which can cause significant damage to foundations and structures due to total or differential settlement. Excessive settlement may also cause catastrophic damage to lifelines
(Cubrinovski and Robinson 2016). Structures on shallow foundation elements are especially prone to experiencing significant settlement due to liquefaction of shallow liquefiable soils. Only once the pore pressures have dissipated and the soil regained its effective stress can applied loads be resisted.

A common indicator of liquefaction within a soil profile is the excess pore pressure ratio, \( r_u \). This ratio is calculated as:

\[
ru = \frac{\Delta u}{\sigma'\varepsilon_vo}
\]

Equation 2-1

where \( \Delta u \) is the instantaneous excess pore water pressure minus the static pore pressure and \( \sigma'\varepsilon_vo \) is the initial vertical effective stress (Rollins et al. 2001). A ratio can be calculated for each discrete depth of interest, thus providing a profile showing where liquefaction has likely occurred.

Liquefaction predominately occurs in saturated sands and silty sands. Even when an excess pore pressure ratio of 1.0 is reached in cohesive soils, liquefaction does not generally produce a loss of shear strength due to the cohesive interparticle forces within the soil (Tokimatsu and Yoshimi 1981). However, some studies have shown that a few low plasticity soils such as silty clays may lose strength from liquefaction under large-magnitude cyclical forces (Carraro et al. 2003; Idriss and Boulanger 2006; Perlea 2000). Thus, when conducting liquefaction susceptibility analysis, it is important to include effects from all types of soil present at the site of interest.

Ishihara and Yoshimine (1992) found in their testing of sands that regardless of the loading conditions (i.e., the direction and magnitude of the load), the volume change characteristics of the samples during reconsolidation following the cyclic loading were directly correlated with the excess pore water pressure ratio that developed. However, this only held true for pre-liquefaction conditions. Therefore, they noted that the amplitude of the maximum shear strain induced during loading would be an appropriate parameter influencing volumetric strain in the subsequent stage.
of consolidation. Figure 2.3 shows the curves they created based on their cyclic laboratory tests which can be used to estimate post-liquefaction volumetric strain based on the relative density of the soil and a given factor of safety for liquefaction. These curves were used to predict settlements and compared with case studies, resulting in acceptable estimates when compared to field data. Other correlations have been made with parameters such as the corrected SPT blowcount \(N_1\) and normalized cone tip resistance \(q_{c1}\). The volumetric strain significantly decreases with increased relative density, CPT \(q_{c1}\), and SPT \(N_1\). Additionally, as the factor of safety for liquefaction decreases below 1.0, strains increase at a staggering rate. For soils with higher relative density, lower factors of safety are required to produce the same maximum shear strain.

While the method developed by Ishihara and Yoshimine (1992) is reasonable for clean sands, it is difficult to perform for an entire soil profile, especially if the profile contains some amount of fines. Continuing on the work of Ishihara and Yoshimine, Zhang et al. (2002) developed curves for volumetric strain based on the equivalent clean sand normalized cone tip resistance, \((q_{c1}N)_{cs}\), from Robertson and Wride (1998). Using this parameter to predict strains accounts for any fines in the soil profile. For sites with level ground, it is reasonable to assume that little or no lateral displacements will occur, resulting in the volumetric strain being equal or close to the vertical strain, allowing for the calculation of liquefaction-induced settlement.

Using the Zhang et al. (2002) approach, the post-liquefaction volumetric strain can be found using the set of curves shown in Figure 2.4 with the accompanying equations for the different factors of safety provided in Table 2.1. The equivalent clean sand normalized cone tip resistance, as described in section 2.3, is coupled with the factor of safety for liquefaction using the Seed and Idriss (1971) simplified approach with triggering curves presented by Youd et al. (2001).
Following the calculation of volumetric strain based on the CPT data, the total liquefaction-induced reconsolidation settlement throughout the soil profile can be computed by:

\[ S = \sum_{i=1}^{j} \varepsilon_{vi} \Delta z_i \]

Equation 2-2

where \( \varepsilon_{vi} \) is the post-liquefaction volumetric strain for the soil sublayer \( i \), \( \Delta z_i \) is the thickness of sublayer \( i \), and \( j \) is the total number of soil sublayers (Zhang et al. 2002). Data from multiple locations affected by the 1989 Loma Prieta earthquake was used by Zhang et al. (2002) to calculate settlements using this procedure and compared against measured values. Good agreement was found between these settlements, indicating that the CPT-based method may be used to estimate liquefaction-induced settlements for low to medium risk projects.

Figure 2.3: Chart for determining volumetric strains as functions of factor of safety (Ishihara and Yoshimine 1992).
Figure 2.4: Relationship between post-liquefaction volumetric strain and equivalent clean sand normalized CPT tip resistance for different factors of safety (Zhang et al. 2002).

The Zhang et al. (2002) method was used in this thesis for estimating the liquefaction-induced volumetric strain and resulting settlements in the natural panel and the improved panel. As shown in Table 2.1, the equations for calculating volumetric strains are only provided for FS_L intervals of 0.1 between 0.5 and 2.0. Using these equations for a discrete FS_L of 1.0, good agreement was found between the calculated settlements and the measured settlements in the IP and NP, which will be discussed in greater detail in section 7.3.4.
Table 2.1: Relationship between post-liquefaction volumetric strain and \((q_{\text{c1N}})_{cs}\) for various factors of safety (Zhang et al. 2002).

<table>
<thead>
<tr>
<th>FS</th>
<th>(\varepsilon_v)</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>(102(q_{\text{c1N}})<em>{cs}^{0.82}) for 33 (\leq (q</em>{\text{c1N}})_{cs} \leq 200)</td>
<td></td>
</tr>
<tr>
<td>0.6</td>
<td>(2411(q_{\text{c1N}})<em>{cs}^{-1.45}) for 147 (\leq (q</em>{\text{c1N}})_{cs} \leq 200)</td>
<td></td>
</tr>
<tr>
<td>0.7</td>
<td>(102(q_{\text{c1N}})<em>{cs}^{0.82}) for 110 (\leq (q</em>{\text{c1N}})_{cs} \leq 200)</td>
<td></td>
</tr>
<tr>
<td>0.8</td>
<td>(1701(q_{\text{c1N}})<em>{cs}^{-1.42}) for 80 (\leq (q</em>{\text{c1N}})_{cs} \leq 80)</td>
<td></td>
</tr>
<tr>
<td>0.9</td>
<td>(102(q_{\text{c1N}})<em>{cs}^{0.82}) for 60 (\leq (q</em>{\text{c1N}})_{cs} \leq 60)</td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>(1430(q_{\text{c1N}})<em>{cs}^{-1.48}) for 60 (\leq (q</em>{\text{c1N}})_{cs} \leq 200)</td>
<td></td>
</tr>
<tr>
<td>1.1</td>
<td>(64(q_{\text{c1N}})<em>{cs}^{0.93}) for 33 (\leq (q</em>{\text{c1N}})_{cs} \leq 200)</td>
<td></td>
</tr>
<tr>
<td>1.2</td>
<td>(11(q_{\text{c1N}})<em>{cs}^{-0.65}) for 33 (\leq (q</em>{\text{c1N}})_{cs} \leq 200)</td>
<td></td>
</tr>
<tr>
<td>1.3</td>
<td>(9.7(q_{\text{c1N}})<em>{cs}^{0.69}) for 33 (\leq (q</em>{\text{c1N}})_{cs} \leq 200)</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>(7.6(q_{\text{c1N}})<em>{cs}^{-0.71}) for 33 (\leq (q</em>{\text{c1N}})_{cs} \leq 200)</td>
<td></td>
</tr>
</tbody>
</table>

2.3 Liquefaction evaluation by CPT

At the test site in Christchurch, New Zealand, in-situ data was obtained by CPT and DMT. Initial pre- and post-treatment CPT and DMT data were acquired in 2016 by Mainmark Ground Engineering, with additional CPT readings taken in November of 2019 by researchers from the University of Auckland. Liquefaction analysis was performed using CPT-based methods. As such, only the CPT method presented by Youd et al. (2001) will be discussed, along with various updates by other researchers.

CPT provides a nearly continuous profile of penetration resistance, allowing for the development of a stratigraphic interpretation. Soil layers within the profile can be easily detected and defined by the test, facilitating the creation of liquefaction-resistance profiles. Along with data from physical sampling to corroborate the results of the CPT, it is an extremely powerful test.
Based on the CPT data, the factor of safety against liquefaction can be calculated. The variables, coefficients, and equations necessary to calculate the cyclic stress ratio (CSR), the cyclic resistance ratio (CRR), and the factor of safety against liquefaction (FSL) will be reviewed in this section.

To begin, we calculate the normalized equivalent clean sand cone penetration resistance, \((q_{c1N})_{es}\), following the steps from Robertson and Wride (1998). First the cone tip resistance is normalized to atmospheric pressure, resulting in a dimensionless cone penetration resistance, \(q_{c1N}\) using the equation

\[
q_{c1N} = C_q \left(\frac{q_c}{P_a}\right)
\]

Equation 2-3

where:

\[
C_q = \left(\frac{P_a}{\sigma'_{vo}}\right)^n < 1.7
\]

Equation 2-4

and \(C_q\) is the normalizing factor for cone penetration resistance, \(P_a\) is one atmosphere of pressure in the same units used for \(\sigma'_{vo}\), \(n\) is an exponent that varies with soil type, and \(q_c\) is the cone penetration resistance as measured by the CPT. The purpose of the overburden normalization is to obtain quantities that are independent of effective vertical stress and thus more uniquely related to the soil’s relative density.

Although there have been many discussions on what value to use for the exponent, \(n\), the equation developed by Robertson (2009)

\[
n = 0.381(I_c) + 0.05 \left(\frac{\sigma'_{vo}}{P_a}\right) - 0.15 \leq 1.0
\]

Equation 2-5

will be used in this thesis as it accounts for the variation of the stress exponent with both normalized soil behavior type index (SBTn Ic) and effective vertical stress.

One major benefit of the CPT is being able to provide an understanding of the mechanical characteristics of the soil, such as the strength, stiffness, and compressibility. The soil behavior
type index, $I_c$, provides a simple guide to the continuous variation of soil behavior type in a given soil profile based on CPT results. It is calculated as:

$$I_c = ((3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2)^{0.5}$$  

Equation 2-6

where:

$$Q_{tn} = \left(\frac{q_c - \sigma_{vo}}{P_a}\right) C_q$$  

Equation 2-7

$$F_r = \left(\frac{f_s}{q_t - \sigma_{vo}}\right) 100\%$$  

Equation 2-8

and $f_s$ is the sleeve friction resistance, $F_r$ is the normalized friction ratio, and $q_t$ is the corrected cone resistance. The parameter $Q_{tn}$ represents the simple normalization with a stress exponent, $n$, of 1.0, which applies well to clay-like soils. By plotting the normalized cone resistance against the normalized friction ratio, the SBT can be identified graphically (see Figure 2.5).

Robertson and Wride (1998) recommended when calculating $I_c$ to first differentiate between sand-like and clay-like soils. This differentiation is performed by assuming a stress exponent, $n$, of 1.0 and calculating $Q_{tn}$ using Equation 2-7. In most cases, when the resulting SBT index is greater than 2.6, the soil is classified as clayey and therefore non-liquefiable. Corroboration from soil samples is suggested if this is the case, as there are still specific instances where the soils could be susceptible to liquefaction. If the calculated $I_c$ is less than 2.6, the soil is likely to be granular in nature. Therefore, $C_q$ and $Q_{tn}$ should be recalculated using the stress exponents based on Equation 2-5 and then used to find $I_c$. If this $I_c$ is less than 2.6 the soil is classed as non-plastic, granular and is used to estimate liquefaction resistance.
To account for the fines content and plasticity of the soil, a correction factor for grain characteristics, $K_c$, is applied. This correction factor is multiplied by the normalized penetration resistance $(q_{c1N})$, which results in an equivalent clean sand value, noted by Equation 2-9. Equation 2-10 and Equation 2-11 are used to calculate $K_c$ based on the value of the SBT index.

$$ (q_{c1N})_{cs} = K_c q_{c1N} $$  \hspace{1cm} \text{Equation 2-9} \\

for $I_c \leq 1.64$, $K_c = 1.0$  \hspace{1cm} \text{Equation 2-10} \\

for $I_c > 1.64$, $K_c = -0.403l_c^4 + 5.581l_c^3 - 21.631l_c^2 + 33.75l_c - 17.88$  \hspace{1cm} \text{Equation 2-11} \\

Once the $(q_{c1N})_{cs}$ values are calculated, they can be used to calculate the post-liquefaction strains and resulting settlements as described in section 2.2 and the CRR described in this section. The ground improvement process may alter the post-treatment $I_c$ value, but the improvement
process does not fundamentally change the soil or alter the fines content, therefore, it is often necessary to use the pre-improved $I_c$ for post-improvement calculations when evaluating the effectiveness of the improvement (Nguyen et al. 2014).

The cyclic shear stress amplitude for level ground used to compute the $FS_L$ is calculated using the simplified procedure presented by Seed and Idriss (1971) that was subsequently updated by Idriss and Boulanger (2006). It is represented as:

$$
\tau_{cyc} = 0.65 \frac{a_{max}}{g} \sigma_v r_d
$$

where $a_{max}$ is the peak ground acceleration in units of g’s, $\sigma_v$ is the total vertical stress, and $r_d$ is the stress reduction coefficient, which is a function of depth and earthquake magnitude, given by the following equations (which are applicable for depths less than 34 m):

$$
r_d = e^{a(z)+\beta(z)\cdot M}
$$

$$
a(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right)
$$

$$
\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right)
$$

where $z$ is the depth in meters, $M$ is the moment magnitude of the earthquake, and the expression inside of the parentheses are in radians. The factor of 0.65 is used to convert the peak cyclic shear stress ratio to a cyclic stress ratio that is representative of the most significant cycles over the full duration of loading (Idriss and Boulanger 2006). The stress reduction coefficient may also be found graphically using Figure 2.6, though for magnitudes other than those plotted it may be easier to use the respective equations.
To standardize the calculations of FS_L, it is customary to adjust the equivalent number of stress cycles to pertain to the equivalent uniform shear stress induced by ground motions generated by an earthquake of M_w = 7.5 (Idriss and Boulanger 2006). Thus, the final equation for the adjusted CSR is:

\[
C_{SR_{M=7.5}} = \frac{C_{SR}}{MSF} = 0.65 \left( \frac{\sigma_{v0}a_{max}}{\sigma'_{v0}g} \right) \frac{r_d}{MSF}
\]

Equation 2-16

The magnitude scaling factor (MSF) proposed by Idriss (1999) is calculated as:

\[
MSF = 6.9 \exp \left( \frac{-M}{4} \right) - 0.058 \leq 1.8
\]

Equation 2-17

where M is the moment magnitude of the earthquake.

The cyclic resistance ratio is the cyclic stress ratio that causes liquefaction for a M_w = 7.5 earthquake as obtained from the case-history-based semi-empirical correlations (Idriss and Boulanger 2006). To calculate the CRR_{M=7.5} the following equations from Robertson and Wride (1998) may be used:
If \( 50 \leq (q_{c1N})_c s \leq 160 \), \( CRR_{7.5} = 93 \left[ \frac{(q_{c1N})_c s}{1000} \right]^3 + 0.08 \) \hspace{1cm} \text{Equation 2-18}

If \( (q_{c1N})_c s < 50 \), \( CRR_{7.5} = 0.833 \left[ \frac{(q_{c1N})_c s}{1000} \right]^3 + 0.05 \) \hspace{1cm} \text{Equation 2-19}

An alternative correlation for CRR put forth by Idriss and Boulanger (2006) is:

\[
CRR_{7.5} = \exp \left\{ \frac{q_{c1N}}{540} + \left( \frac{q_{c1N}}{67} \right)^2 - \left( \frac{q_{c1N}}{80} \right)^3 + \left( \frac{q_{c1N}}{114} \right)^4 \right\} - 3 \]

\hspace{1cm} \text{Equation 2-20}

which is a function of the normalized (to atmospheric pressure) cone tip resistance rather than the equivalent clean sand normalized cone tip resistance. Equation 2-20 is only applicable for cohesionless soils having a fines content \( \leq 5\% \) and is more conservative than the Robertson and Wride equations (Idriss and Boulanger 2006). Two factors are then applied to the CRR before it can be used to calculate the factor of safety against liquefaction: the overburden correction factor, \( K_\sigma \), and the static shear stress correction factor for sloped grounds, \( K_\alpha \) (Boulanger and Idriss 2004). The overburden correction factor is calculated as:

\[
K_\sigma = 1 - C_\sigma \ln \left( \frac{\sigma'_v}{P_a} \right) \leq 1.0 \]

\hspace{1cm} \text{Equation 2-21}

where

\[
C_\sigma = \frac{1}{37.3 - 8.27(q_{c1N})^{0.264}} \leq 0.3 \]

\hspace{1cm} \text{Equation 2-22}

For flat ground, such as the site used for the blast testing in this thesis, \( K_\alpha = 1.0 \). \( CRR_{7.5} \) may also be adjusted to account for the magnitude of a given earthquake using the following equation:

\[
CRR = CRR_{7.5} \times MSF \times K_\sigma \times K_\alpha \]

\hspace{1cm} \text{Equation 2-23}

Once the CSR_{7.5} and CRR_{7.5} have been calculated based on the above equations, the factor of safety against liquefaction may be calculated as:
A factor of safety greater than 1.2 indicates that the soil is unlikely to generate an \( r_u \) greater than about 20% (Marcuson III et al. 1990) or volumetric strains more than 0.5% (see Figure 2.8), whereas a factor of safety less than 1.0 indicates probable liquefaction. An example plot of CSR and CRR versus depth is shown in Figure 2.7. The areas shown in red are where the CRR is less than the CSR, signifying that liquefaction in that section of the soil profile is likely to occur during a seismic event. Knowing the \( FS_L \), the expected excess pore water pressure ratio, \( r_u \) can be determined using correlations such as those shown in Figure 2.8, which are based on laboratory data for sand and gravel.

![CRR plot](image)

Figure 2.7: Example plot of CRR and CSR versus depth to determine where liquefaction may occur.
Figure 2.8: Typical relationships between residual excess pore pressure ratio and FS_L (Marcuson III et al. 1990).

2.4 Summary

In the past few decades, much has been learned about liquefaction from both laboratory and case studies. Theoretical and empirical methods have been developed to predict the initiation of liquefaction and the magnitude of liquefaction-induced settlement. However, even with all the advances made in the geotechnical field, there is still much to be learned about this phenomenon. Most of the existing relationships were derived from data of existing case studies, using measured data to estimate soil parameters of the soils from the studies. With an increasing number of earthquake events, more data will become available for researchers to further refine the relationships to reflect reality more accurately and to better understand the phenomenon of liquefaction. This study will add to the collection of data available for such analyses.
3 LIQUEFACTION MITIGATION AND RESIN INJECTION TREATMENT

Despite the advances in the understanding of geotechnical engineers with regards to earthquakes, destruction of life and property continues to occur each year. The continued study of ground improvement methods is fueled by the need for safer and more reliable methods. Many mitigation techniques exist which seek to improve the soil by increasing the strength and stiffness. Others attempt to mitigate the buildup of pore water pressures, as they are related to the strain potential of the soil. Regardless of the mechanism by which they improve the ground, these methods are used at sites where the existing soil conditions are expected to lead to unsatisfactory performance, typically in the form of unacceptably large soil movements.

3.1 Existing mitigation techniques

A wide range of soil improvement techniques are available for eliminating undesirable aspects of the soil. This section will only discuss those techniques that are relevant to mitigating seismic hazards. Four main categories of ground improvement mitigation techniques exist: reinforcement, grouting and mixing, densification, and drainage (Kramer 1996). While methods can be organized into these categories in a general sense, many include aspects of multiple improvement methods.

Reinforcement techniques function through installing discrete inclusions that reinforce the surrounding soil. Materials for these inclusions may consist of structural materials such as steel,
concrete, timber, or geomaterials such as densified gravel. Stone columns are a popular reinforcement technique in which dense columns of gravel are installed in native soil. There are many installation methods for stone columns, but all result in increased shear strength, increased rate of radial drainage, and the presence of the stronger stone material (Barksdale and Bachus 1983). Driven piles are another reinforcement technique, in which piles of timber or prestressed concrete are driven into the ground in a grid pattern. Installing the piles improves the ground through densification and increased lateral stresses caused by the presence of the pile. The piles themselves also provide improved seismic performance of the soil through their flexural strength (Gianella 2015). Lastly, drilled inclusions such as drilled shafts have been used to stabilize soils, though the difficulty with these occurs during the installation process when the soil is granular and loose, as they depend on cohesion from the surrounding soil to avoid collapse of the boring prior to concrete placement (Fang et al. 2014). In most cases, a temporary casing or drilling slurry is used to prevent collapse of the borehole prior to placement of the concrete.

Grouting and mixing techniques work through injecting or mixing cementitious material into the soil. Minimal vibrations and settlements result from these methods, making them desirable for situations where strict limits are in place for disruption of the improvement site. Grouting techniques fill the void space and fractures in the soil matrix. Two primary types of grouting techniques are permeation grouting and intrusion grouting. The first uses a low-viscosity liquid grout which fills the voids in the soil without damaging the soil structure, which reduces the tendency for densification and buildup of excess pore water pressure. The latter utilizes pressure to cause controlled fracturing of the soil along weak bedding planes, thus densifying and stiffening the soil. Unlike permeation grouting, intrusion grouting is not intended for filling the voids between soil particles (Littlejohn 2003).
Mixing techniques introduce cementitious material by physically mixing the soil which disturbs the soil structure. Soil mixing is one technique which involves using augers to thoroughly mix the grout into the soil as the augers are advanced into the soil. Once the desired depth is reached, the augers are withdrawn while the mixing process continues, leaving behind a uniform-width column of soil-cement (Topolnicki 2004). Another technique is jet grouting, where an injection nozzle is inserted into a borehole and grout is placed in all directions under high pressure. As the nozzle is lifted, it continues to rotate, leaving behind a relatively uniform column of mixed soil-cement (Burke 2004). With both soil mixing and jet grouting, the columns of soil-cement can be overlapped to create a wall or cellular structure that may remain as part of the permanent structure. For this case, the columns or cellular structure may also serve a reinforcement function.

Densification of soil is a technique used to increase the strength and stiffness of the soil. When the soil is dense, the tendency to generate positive excess pore water pressures due to cyclic loading is lower than when the soil is loose and the potential for volumetric strain is also reduced as previously noted. Because of this, densification is one of the most common and effective means of ground improvement for mitigation of seismic hazards. While densification is very effective, it is not suitable for all sites, particularly those with existing infrastructure, because it leads to permanent volumetric changes that often result in settlement, which is typically undesirable for existing structures.

A common method of densification is vibration, which can take place horizontally or vertically. Horizontal vibration, commonly known as vibroflotation, makes use of a vibroflot to locally liquefy a zone of soil as it is withdrawn. This can result in surficial settlement, which is typically made up for by placing granular material to further strengthen the soil and level the ground. Vertical vibration makes use of a vibro rod system attached to a vibratory pile driving
hammer to insert the probe into the ground and withdraw it as it vibrates. As with vibroflotation, vertical vibration can cause surface settlement so the same method of adding compensatory granular materials at the surface may be used. Because of the smaller areal influence of the vibro rod, the grid spacing must be tighter than that for vibroflotation (Brown and Glenn 1976).

Another method of densification is dynamic compaction, where large weights are repeatedly dropped from a high height, thus imparting energy into the ground and densifying the soil. Dynamic compaction occurs in a grid pattern and multiple stages may occur, as the effective depth of influence of the process depends on the impact energy. Between stages of compaction, granular material may be placed to fill in the craters that are created by this process (Lee and Gu 2004).

Blast densification is a way to temporarily liquefy soils, causing the pore water to be expelled from the soil matrix. Following the dissipation of the water, the ground surface settles as the soil particles densify thus increasing their strength. This method is useful for open sites where disturbances would not be problematic (Narsilio et al. 2009). It has also been used to treat liquefiable layers at depths greater than 20 m where other surficial densification methods may be less effective due to the dissipation of energy with depth (Solymar and Mitchell 1986).

Compaction grouting is a method in which very low slump grout is injected into the soil under high pressure. A resulting bulb or column of grout is created, thus displacing and densifying the native soil. At shallow depths, compaction grouting can be used to lift settled slabs or structures as the high pressure of the grout causes ground heaving at the surface (Miller and Roycroft 2004).

The last category of mitigation techniques deals with the drainage of water within a soil profile. As liquefaction commonly occurs in saturated soils due to the generation of excess pore water pressures, adding the ability for the water to drain from the soil instead of building up
pressure can mitigate the effects (Morikawa and Cho 2020). Dewatering is one way to lower the groundwater table. However, due to the high cost and limited long-term efficacy of dewatering, it is not recommended. Similarly, gravel drains can be installed to allow for drainage, though these can become clogged with fines over time limiting their efficiency (Seed and Booker 1977). In general, drainage techniques may not be the best option for mitigating seismic hazards, as post-earthquake settlements can still occur, and the systems often require frequent maintenance.

3.2 Resin injection as a method of ground improvement

A final form of ground improvement technique will be discussed, which is the focus of this research. The Teretek® resin injection solution system is a proprietary ground improvement technique which was developed by Mainmark Ground Engineering (NZ) Ltd. However, similar resin injection strategies are provided by other geotechnical specialty contractors throughout the world.

Resin injection has long been used as a method for re-levelling structures following ground subsidence. Because of the non-intrusive nature of the installation process, it can be used under existing structures to mitigate liquefaction potential (Traylen et al. 2018) where other traditional ground improvement mitigation techniques would not be viable. Recent studies have shown that it also provides ground improvement characteristics and can mitigate liquefaction potential.

The main method of improvement comes from soil densification, similar to compaction grouting. Unlike compaction grouting, the resin injection process does not produce regular bulbs or columns of material down the vertical injection line. Instead, it typically results in a vein-like structure of material distributed throughout the soil mass in the form of dykes, sills, or networks of sheets or planes as shown in Figure 3.1. These are usually tens of millimeters in thickness (Hnat et al. 2017).
In this section the construction methodology of the resin injection is described and the current and past research that has reviewed the mechanisms associated with this process is explored.

### 3.2.1 Construction methodology

With the Teretek resin injection system, 16 mm-diameter (as small as 6 mm depending on the project) injection tubes are driven into the ground in regularly spaced intervals, typically in a triangular or square grid pattern. An injection nozzle is attached to the injection tube to allow for mixing of the proprietary resin constituents at specific pressures and temperatures. The composite material, i.e., the resin, is then pumped down to the base of the tube where it enters the soil matrix. Both top-down (downstage) and bottom-up (upstage) methods can be used for the treatment process.

In a typical bottom-up installation, the injection tube is driven down to the target depth and then withdrawn either in set stages with set volumes of resin material being injected per unit length.
of withdrawal, or it is slowly withdrawn at a uniform rate with set volumes of resin material being injected per unit length of withdrawal. This approach tends to be more efficient than the top-down method with respect to installation time and cost of injection hardware (not the resin itself).

In a typical top-down installation, the injection tube is driven down while set volumes of resin material are injected at discrete depths, or while injecting a set volume of resin material per unit length while being continuously driven down. This approach has the advantage of being able to provide a capping layer at the beginning of the process, which can help reduce the likelihood of ground heave during injection by contributing additional confinement and strength to loose soils.

This process can be carried out under existing structures, one of the main advantages of this ground improvement method over other existing methods. However, it can also be done in a “free field” situation where no overlying obstructions are present. In this case, it is common practice to add a surcharge load across the site to prevent ground heave. Additionally, controlling the reaction and expansion characteristics of the resin material can be accounted for to avoid the issue of ground heave as well.

Once pumped through the injection tubes, the resin penetrates the soil mass along pre-existing planes of weakness or through fracturing the soil matrix, much like intrusion grouting. The resin also permeates the soil mass, to a limited extent, depending upon the porosity of the soil, similar to permeation grouting. Having a higher fines content decreases the ability of the resin to permeate the soil matrix. Soon after injection the resin rapidly expands to many times its original volume, changing from a fluid form to a solid one. The change in volume can be on the order of 5 to 15 times the injected volume. This depends largely on the soil density, confinement pressures, and the specific resin material used for the application. Typically, looser sandy soils allow for
greater expansion of the resin mixture. The expansion process drives the densification of the soils surrounding the injected resin.

The depth at which the resin is injected depends on the location of the site being treated. This process is mainly used for strengthening the soil under shallow foundations, whether it be concrete slabs on grade or below an exterior footing and foundation wall system. Typical depths of treatment range from 1 to 7 m below grade.

3.2.2 Current state of resin injection ground improvement research

Polyurethane resin injection has long been used as a method of re-levelling buildings after differential settlement or earthquake-induced subsidence (Erdemgil et al. 2007; Mainmark 2020; Sánchez et al. 2017; Van Reenen 2006). An additional application of this ground improvement technique includes strengthening weak soils below roadways (Popik et al. 2010; Yu et al. 2013). As noted by Kramer (1996), most ground improvement methods begin being implemented prior to the theoretical framework of the improvement mechanism being vetted by researchers. This is true for the resin injection ground improvement, as it began to be used years ago for re-levelling buildings and is now being touted as a form of liquefaction mitigation.

Resin injection has been used in a wide variety of geotechnical conditions to improve the soils characteristics, such as expansive clayey soils (Buzzi et al. 2010), alluvial deposits (Erdemgil et al. 2007), organic soils (Popik et al. 2010), and sandy soils (Traylen et al. 2018). While not all these soil types are susceptible to liquefaction, they have all benefited from the resin injection treatment to improve the soil properties. The ability to be used in such a wide range of soil types is beneficial, as many ground improvement methods are limited to distinct classifications of soil due to their improvement mechanisms.
Many lab and small-scale tests have shown that polyurethane resin injection treatment is effective in treating soil to mitigate liquefaction potential. Prabhakaran et al. (2020) conducted a shake table test in which the dynamic response of the soil foundation system and the polymer-sand composite were examined. They observed a marked increase in resistance to liquefaction due to densification. Additionally, a softened response to the acceleration of the shake table was noted, signifying the presence of a stiffer soil stratum post-injection. As liquefaction at shallow depths typically results in surface settlement, a stiffer crust at the top of the profile can reduce the potential for damaging differential settlements (Wentz et al. 2015).

Sabri et al. (2018) tested the effectiveness of the resin beneath a concrete foundation and found that increased bearing capacity, decreased settlement, and an increase in deformation modulus resulted. Similarly, Traylen et al. (2018) and Hnat et al. (2017) reported on ground improvement that occurred beneath a commercial shopping center and a retirement village. Post-improvement testing showed significant increases in CPT cone resistance, increased relative density, and DMT horizontal stress index. The effects of the treatment were most notable in sandy soils, though some soils with higher fines contents also benefitted. Both studies highlighted the importance of being able to improve the ground beneath existing structures that must remain functional during the improvement process. With traditional methods of ground improvement, this may not be possible due to the vibration, noise, or deformation required to carry out the improvement process. However, the non-intrusive nature of the resin injection treatment allows for business to continue as usual during the treatment. Resin ground improvement has also been used to improve soil conditions beneath historical structures, such as the palace of Cardinal Diego de Espinosa located in Segovia, Spain (Sánchez et al. 2017). This process can also be used in free-
field settings prior to construction to mitigate any potential hazards that exist at the site; however, competing ground improvement techniques may be more economical for this situation.

Few full-scale tests have been conducted to test the efficacy of resin injection treatment at present. One test, conducted by Apuani et al. (2015) employed three-dimensional electrical resistivity tomography (3D ERT) using a pole-dipole array to produce 3D images of the soils before, during, and after resin injection. Through this process, real-time evaluation of the densification process caused by the resin was possible. The sensitivity of the ERT to the presence of water or voids allows for accurate mapping of the instrumented profile. Paired with the results from traditional geotechnical tests, an increase in mechanical resistance, stiffness, and bearing capacity was observed. Figure 3.2 shows the pre- and posttreatment test results at a depth of 125 cm below the ground surface, where marked increases in resistivity and cone penetration resistance are readily observed, signifying the efficacy of the treatment. Additionally, by means of ERT, they noted that as the resin was injected the water in the treatment area migrated from the injected volumes because of the densification process. Though the displacement of the water was not permanent, it confirmed the ability of the resin to densify the soils local to the injection points. With real-time monitoring provided by means of 3D ERT, anomalies in the resistivity of subsurface soils can be identified to locate areas in need of improvement, thus allowing for the optimization of the injection plan to minimize treatment cost and investigation time (Santarato et al. 2011).

Another full-scale experiment by Van Reenen (2006) measured the effectiveness of resin injection under a strip footing loaded with surcharge. Though CPT and dynamic sounding did not reveal marked improvement in soil strength when compared to untreated soil, densification was achieved through the treatment process. At one of the sites from the study, a trench was dug to
examine the spread of the resin within the soil matrix. As expected, the resin followed the path of least resistance along pre-existing fractures, creating a dendritic network of resin similar to that reported by Traylen et al. (2017). If two different soil types were adjacent, the resin followed the border along the interface in the weakest soil. The maximum horizontal spread within the soil was 2.5 – 3 m, confirming that a spacing of 1.2 m center-to-center as noted in section 5.1 would allow for some overlap between injections. Thicknesses of the resin veins were on the order of 10 mm, which may have been larger were it not for the loss of heat due to groundwater retarding the expansion process.

Figure 3.2: Comparison of (a) pre- and (b) posttreatment test results: horizontal section of the 3D resistivity model at a depth of 125 cm and Qc contour lines by interpolation (Apuani et al. 2015).
With traditional methods of ground improvement there are typically empirical equations and correlations that can be used to predict the improvement of the treated soil. For resin injection, no reliable methods of predicting the improved characteristics of the soil exist. Sabri and Shashkin (2020) attempted to model the soil-resin interaction using the finite element method (FEM) based on the results of earlier plate load tests obtained from field experiments. A triaxial test model was created and calibrated, which later included different configurations of the resin within the soil to account for different injection and propagation patterns of the resin. Each configuration resulted in slightly different stresses and strains within the soil, though not too dissimilar from one another. Additional plate load tests were simulated to corroborate the results of the triaxial model. Despite the development of a quasi-empirical method for evaluating and predicting post-improvement changes, the limited testing and corroboration to real test results leads to large uncertainty in the accuracy of this method.

One important property of polyurethane resin is its longevity. Unlike metals, concretes, and woods, resin grout is not subject to much, if any, deterioration with time. Accelerated and natural aging tests have been carried out to determine the effects of time on resin injection (Oshita et al. 1991; van der Wal 2010). Factors affecting aging effects include mechanical stresses, heat, moisture, cyclic temperatures, biodegradation, and radiation exposure. Based on aging studies and industry experience, the expected lifetime of the treatment is around 50 years (Mainmark 2020). The inert nature of the resin injection also provides an additional benefit to the treatment, as it does not have any detrimental effect on the environment due to decomposition or degradation, does not affect groundwater, or soil quality (Personal communication, Rex Klentzman, Uretek, 2022).

In summary, polyurethane grouts have been used below existing structures as well as in free-field conditions to improve the ground and mitigate liquefaction potential. Increased bearing
capacity and reduced liquefaction-induced settlement resulting from densification by resin injection are the most notable effects of the treatment. No viable methods of predicting improved conditions are presently available, so the findings of case studies must be relied upon. As more is learned about this technology and its advantages, it will likely become more popular with contractors and engineers alike (Naudts 2003).
4 PRELIMINARY INVESTIGATIONS AND SITE CHARACTERIZATION

4.1 Geotechnical conditions

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. These included rapid impact compaction, geopier rammed aggregate pier reinforcement, driven timber piles, low mobility grout, resin injection, reinforced gravel rafts, reinforced soil-cement rafts, and horizontal soil-cement mixed beams. 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 (Commission 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).

Three sites were selected to use for the RZT, testing the efficacy of the resin injection ground improvement. Each site was located away from the 2013 EQC trial sites to ensure that they were not affected by the installation of previous ground improvements and instrumentation (Traylen et al. 2017). Blast testing for this research occurred at Site 3 (Breezes Road, Avondale)
in Christchurch, New Zealand. Prior to the devastating 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 (see Figure 4.1). The location of the test site is shown in yellow for both satellite images, which is located approximately 100 m east of the Avon River.

At Site 3, the soil site characterization was determined using cone penetration tests with pore pressure measurements (CPTu or piezocone test), geophysical tests, downhole characterizations, a plate load test, and seismic dilatometer testing (SDMT). A summary of the testing schedule is provided in Table 4.1. Data from CPTu and SDMT were obtained in 2016 before and after resin injection ground improvement by Mainmark Ground Engineering, with additional in-situ testing being performed throughout the month prior to the blast testing.

The general soil conditions based on these tests consist of silty sand with occasional silt bands to 2.5-3.0 m below ground level. Underlaying this is a fine to coarse sand to 8 m depth, with 0.5 m of silty fine sand below. From 8.5-15 m (maximum depth of CPTu and SDMT soundings), the profile is again a fine to coarse sand. Soil profiles for the NP and the IP are generally the same, with small variations in the location of the silt bands. Laboratory testing of a silty sand sample from the upper layer in the pre-improved IP showed fines content of approximately 40%, while the deeper, cleaner sand layer had fines contents of only 1 to 3%.
Figure 4.1: Residential neighborhood in September 2010 (left) and the same area in November 2015 (right) following the CES.

Table 4.1: Site 3 IP testing schedule

<table>
<thead>
<tr>
<th>Test</th>
<th>Number carried out</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pre-injection</td>
</tr>
<tr>
<td>Cone penetration test</td>
<td>4</td>
</tr>
<tr>
<td>Cross-hole geophysical tests (Vs, Vp)</td>
<td>2</td>
</tr>
<tr>
<td>Dilatometer tests</td>
<td>2</td>
</tr>
<tr>
<td>Plate load test</td>
<td>2</td>
</tr>
<tr>
<td>Borehole</td>
<td>1</td>
</tr>
<tr>
<td>Fines content lab test</td>
<td>4</td>
</tr>
<tr>
<td>Plasticity index lab test</td>
<td>1</td>
</tr>
</tbody>
</table>

The results from the CPTu and SDMT in-situ testing at the NP and IP prior to the resin installation are plotted in Figure 4.2 and Figure 4.3, respectively. The $I_c$ profile from the CPT plot shows that the top 2.5 meters primarily consists of sandy mixtures (silty sand to sandy silt) which are underlain by clean sands. The SDMT indicates a similar trend, though the profile switches between silty sand and sandy silt for the depth of the reading.
Figure 4.2: Comparisons of CPTu test results at the NP and the IP prior to resin injection with respect to (a) cone resistance, $q_c$, (b) sleeve friction, $f_s$, (c) porewater pressure, $U_2$, (d) soil behavior type, $I_c$, (e) ratio of constrained modulus to cone tip resistance, $M/q_c$, and (f) interpreted soil profile.

The parameters obtained from the CPTu as shown in Figure 4.2 are mostly similar, with the exception of the location of the silt bands within the profile. Figure 4.3 also shows that the SDMT survey of the two profiles generally provides good agreement between these locations in the upper 6 m, although the DMT-based constrained modulus is about 30 to 40% higher from 3 to 5 m in the NP and both the $K_D$ and constrained modulus are more than 100% higher in the NP from 5 to 6 m. As the 2016 DMT readings did not go past 8 m, additional DMT tests were performed the week prior to the blast testing in the natural and improved panels to a depth of 15 m. This testing was conducted by Dr. Sara Amoroso of the University of Chieti-Pescara, with the resulting data plotted in Figure 4.4. Between 7 to 9 m, a significant increase in the NP’s stiffness as denoted by the horizontal stress index and constrained modulus is observed when compared to
the IP. From 9 to 12 m the inverse occurs, and at 12 m the parameters in both panels become relatively uniform until the ultimate depth of 15 m is reached.

![Figure 4.3: Comparisons of SDMT test results at the NP and the IP prior to resin injection with respect to (a) horizontal stress index, $K_D$, (b) soil material index, $I_D$, (c) constrained modulus, $M$, (d) fines content, $FC$, (e) shear wave velocity, $v_s$, and (f) interpreted soil profile.](image)

The apparent fines content ($FC$) plot is based on the modified soil behavior type index, $I_c$, as proposed by Robertson and Wride (1998):

\[
\text{if } I_c < 1.26 \text{ then } FC(\%) = 0 \quad \text{Equation 4-1}
\]

\[
\text{if } 1.26 \leq I_c \leq 3.5 \text{ then } FC(\%) = 1.75I_c^{3.25} - 3.7 \quad \text{Equation 4-2}
\]

\[
\text{if } I_c > 3.5 \text{ then } FC(\%) = 100 \quad \text{Equation 4-3}
\]
Figure 4.4: A comparison of the (a) horizontal stress index and (b) constrained modulus between the NP and IP based on December 2019 SDMT testing.
As with the results from the sieve analysis (see Figure 4.5) performed by Central Testing Services, a materials testing contractor, on a sample acquired during drilling on site in August of 2016, the calculated apparent fines content in the top 2 to 3 m of the soil profile is higher than deeper down where the poorly graded sands are located, which is due to the bands of silt mentioned earlier. Based on the gradation curve of the sample from 5 m deep, the soil at that depth is classified as a fine-grained sand. Discrete points from lab testing shown in Figure 4.3(d) display good correlation to the apparent fine content from 3 m down, whereas in the upper 2 m a higher percentage of fines is reported from the sieve analyses than shown by the calculated fines content. The noticeable decrease in fines content around 2.5 m correlates well with the indicated soil type as it transitions from the upper layer of sandy mixtures to the underlying layer of poorly graded sands.

![Grain size curve of soil at Breezes Road test site at a depth of 5 m.](image)

**Figure 4.5:** Grain size curve of soil at Breezes Road test site at a depth of 5 m.
Figure 4.6 shows data from one of the CPTu readings taken in the IP prior to treatment plotted on the normalized soil behavior type chart (Robertson 1990). The top 2.5 m of the soil profile consist of soils with $I_c$ around 2.05 or greater, indicating a sand mixture of silty sand to sandy silt. This agrees with the lab testing, which showed that there were silt bands in the top few meters. Below that, the soils have an $I_c$ of 2.05 or less, indicating that they are clean sands with a few traces of silt.

The depth of groundwater at the site varies with the season. During the CPTu soundings pre- and post-treatment of the IP, the water table was between 0.90 and 1.11 m below the ground surface. The upper bound of the water table was used in calculations for post-liquefaction settlement to be sure that settlement predictions were not too conservative.

![Figure 4.6: Soil behavior type plot consisting of CPTu data from a sounding in the IP prior to treatment.](image-url)
4.2 Liquefaction assessment

A liquefaction potential assessment was performed as part of this study using the Idriss and Boulanger (2006) CPTu method for an earthquake with $M_w = 7.5$ and peak ground acceleration $a_{max} = 0.20g$, which is equivalent to the 100-year return period earthquake in Christchurch, New Zealand (Traylen et al. 2018). This assessment was performed using CLiq v.3.0.3.4, a commercial program used for various liquefaction-related analyses that was developed by GeoLogismiki in collaboration with Gregg Drilling and Dr. Peter Robertson. The resulting analysis of the pre-treated IP soil profile showed liquefiable layers between 1-3.5 m and 5-8 m given the selected magnitude and ground acceleration. The CSR and CRR in the pre-treated IP, based on one of the CPTu soundings, are plotted with depth in Figure 4.7(a).

![Figure 4.7: (a) Cyclic stress ratio (CSR) and cyclic resistance ratio (CRR), (b) Factor of safety against liquefaction in the pre-treated IP.](image-url)
The areas shaded in red are depths where the CRR is less than the CSR, meaning those regions in the soil profile are potentially liquefiable as the $FS_L < 1.0$. Additionally, Figure 4.7(b) shows the $FS_L$ versus depth throughout the profile. When the $FS_L$ is in the region with the green shading, it signifies depths in the profile where liquefaction is not likely to occur. Analyses on each of the CPTu soundings for the pre-treated IP show similar trends in CSR, CRR, and $FS_L$. As the pre-treated CPTu tests only went to a depth of 8 m, it is impossible to tell if liquefiable layers exist below that depth, though later CPTu soundings near the panel show potentially liquefiable layers from 8 to 10 m as well.
5 GROUND IMPROVEMENT PROCEDURE

5.1 Injection of resin improvement

Over the course of a few days at the end of September and beginning of October in 2016, polyurethane resin injection using the Teretek™ system at the test site was performed by Mainmark Ground Engineering (NZ) Ltd and was carried out in accordance with the procedure described in section 3.2.1. This section describes the details of the actual installation performed at the test site.

Prior to resin injection, the surficial topsoil was removed and replaced with a 0.2-m thick layer of compacted gravels to provide a stable working platform. Plywood sheets were then laid over the gravel working surface to serve as a base for the 0.6-m tall concrete blocks, which functioned as a 14 kPa surcharge load representative of a structure that would benefit from the application of resin injection. Welded steel plates were placed over these blocks to give a stable working platform for the contractors. An additional 17 kN/m line load (0.6 m width) was laid across the center of the panel to represent the load of a typical strip footing. The overall purpose of these loads was to better replicate the soil response beneath a building during the resin injection process by providing additional vertical stresses. Figure 5.1 is an aerial photograph taken of the test site in 2017 during the installation of the resin injection at the IP which shows the layout during the installation process as mentioned above.
During the creation of the IP, 50 injection points were used to improve the soil. These injection points were selected to be evenly spaced on a 1.2-m triangular grid, thus creating an improved panel roughly 8 m in diameter. This spacing and configuration was determined by contractors during a pre-production test carried out to refine their installation technique and to verify the effectiveness of the resin in different soil types. Figure 5.2 shows the selected arrangement of the injection points and the location of various in-situ tests performed pre- and post-injection to verify the effectiveness of the improvement process.

![Image of the Breezes Road (Site 3) test site](image)

**Figure 5.1:** Aerial photograph at the Breezes Road (Site 3) test site showing the setup for and installation of resin using the Mainmark Teretek™ system (Traylen et al. 2017).

The installation methodology was the same for each injection point as the contractors had standardized the process during the pre-production testing. Pilot holes were drilled and cored through the steel plate, concrete blocks, and plywood to allow for the installation of the injection tubes into the ground.
The resin injection process consisted of a hybrid method between the top-down and bottom-up processes: injecting a capping layer at a depth of 1 to 1.5 m and then pushing the injection tube to 6 m and withdrawing the tube at 0.5 m increments following the bottom-up method. A set volume of resin was injected at each half-meter increment based on the outcomes of the pre-production test and the contractor’s previous experience from re-levelling buildings.

5.1.1 Ground heave

During the injection process of the IP, 10 to 73 mm of ground heave occurred across the panel, with the average being 37 mm. As the site was to remain in a free-field condition, this heave was expected and therefore not an issue. Approximately 70% of this lift occurred during the injection of the capping layer in the upper profile (Traylen et al. 2017). This cap is intended to restrict the amount of resin that exits the ground surface, rather than expanding more usefully in place at a greater depth.

When used beneath a structure, the heaving of the soil is carefully monitored and controlled to stay within a target range. This can be accomplished by altering the resin’s material characteristics and by changing the sequence of injection points. Differential lift is avoided by temporarily ceasing injection at a point until other adjacent points are brought up to similar levels. In many instances, the structures undergoing this process also require level correction due to subsidence and therefore some ground heave is desirable, so long as it is within the determined tolerance. Where the resin is injected below structures at depths of 2 m or more, no significant ground heave is observed due to the larger vertical confining stresses providing resistance (Traylen et al. 2018).

Unlike using resin injection to treat soils underneath an existing structure, where heaving of the ground could present significant issues, there was no attempt to control ground surface heave
for the IP injections as its effects were not detrimental to the experiments. Even with the heave that occurred, it was still less than that typically observed when improvement methods such as stone columns or driven piles are used.

Figure 5.2: Plan view of test area showing layout of resin injection points along with locations of CPT, DMT, andVs holes (Traylen et al. 2017).

5.2 Post-injection ground improvement evaluation

The equipment required for the installation of the resin injection was removed prior to post-injection ground improvement evaluation, leaving a level and clean surface to work on. In-situ tests were performed at various times after the treatment to compare pre- and post-treatment values and are discussed in this section as a means of quantifying the improvement.
5.2.1 CPT

CPTu data were taken in 2016 prior to the treatment and following the treatment approximately three months later. To better understand how time affects the efficacy of the resin injection treatment, another test was conducted by researchers from the University of Auckland in November of 2019, a few weeks before the blasting occurred. The cone tip resistance, $q_c$, is plotted in Figure 5.4(a) for each of the testing periods. Between the initial test and three months after treatment, the cone tip resistance increased by 105% on average in the treated zone between 1 to 6 m below grade (see Figure 5.3(a)). In the 2019 sounding similar results were obtained, with the average increase within the treated zone being 100% (see Figure 5.3(c)). Knowing that three years after treatment, the soil resistance was at a level similar to that soon after the improvement process, indicates that soil relaxation does not significantly decrease the effectiveness of this treatment over time.

As the overall goal of the resin injection process is to reduce liquefaction potential and subsequent ground subsidence, analyzing the resultant change in $(q_{c1N})_{cs}$ due to the treatment process is important in this respect. The calculations used to predict post-liquefaction settlement are directly correlated to $(q_{c1N})_{cs}$, meaning an increase in $(q_{c1N})_{cs}$ will decrease the predicted settlement. Figure 5.4(b) shows the improvement in $(q_{c1N})_{cs}$ following the ground treatment, and just like $q_c$, the increase in $(q_{c1N})_{cs}$ is still evident three years later.

Figure 5.4(c) shows the different soil behavior types of the soils, though the injection process does not fundamentally change the soil, so these values were not used in calculations that required $I_c$. Rather, the pre-treatment values of $I_c$ were used in lieu of post-treatment $I_c$, using data from CPT tests closest to one another. Indicated by the SBT profile, the top few meters of the site
contained silty sand which was underlain by a fine to coarse sand. A simplified interpreted soil profile across the site is shown in Figure 5.4 (e).

Figure 5.3: CPT cone resistance comparison of (a) before and after improvement, (b) pre-improvement and 2019 natural panel, and (c) post-improvement and 2019 improved panel.
Also shown in Figure 5.4 is the ratio of constrained modulus to cone tip resistance, $M/q_t$. As DMT data from before the injection process were limited to depths between 2 and 6 m, comparisons to post-treatment values can only be compared at those depths. A general trend in the post-improved values shows ratio improvements in the range of 2 to 8. The constrained modulus, $M$, is inversely proportional to ground settlement and typically increases with increasing confinement, which for these tests was provided by the aggressive expansion of the polyurethane resin.

Figure 5.4: Comparisons of CPTu test results at the IP before and after treatment with respect to (a) cone resistance, $q_c$, (b) equivalent clean sand normalized cone tip resistance, $(q_cN)_{eq}$, (c) soil behavior type, $I_c$, and (d) ratio of constrained modulus to cone tip resistance, $M/q_t$, and (e) interpreted soil profile.
The largest amount of improvement occurred in the treated zone of 1 to 6 m. Some improvement was observed between 6 and 6.5 m depths, as the resin most likely expanded and influenced the soil below it as the bottom level of the treatment was injected. Though it is difficult to tell exactly where the influence of the resin ends below the treated zone, it is evident that improvement occurred vertically, not just laterally.

5.2.2 Shear wave velocity, compression wave velocity, and shear modulus

The small-strain shear wave velocity, $V_s$, is an important property of the soil that correlates to the shear modulus, $G$, which is a ratio used to describe a soil’s stiffness. With increasing cyclic shear strain and pore pressures that develop during seismic events, the shear modulus typically decreases. As such, the resulting data obtained from the blast-induced liquefaction testing provides more information that can be added to the repository of knowledge on this topic. Compression wave velocities ($V_p$) at two locations within the IP and initial static values for $V_s$ before and after ground improvement are shown in Figure 5.5. Except for the data recorded in 2019 with the SDMT, all other datasets were obtained via cross-hole geophysical testing performed by Dr. Liam Wotherspoon of the University of Auckland.

A clear increase in shear wave velocity can be seen between the pre- and post-treated plots, with an average increase of 35% in 2016 and 23% in 2019, when compared to pre-improved conditions. Cross-hole geophysical readings were taken for the soil between injection points and across a section of improved soil (i.e., composite resin-soil), and Figure 5.5(d) represents the average of these tests.

Because of the different method (SMDT) used to obtain $V_s$ in December 2019 compared to the other three readings, it is not pertinent to compare against the data from the cross-hole
geophysical testing. Recording SDMT $V_s$ data for each of the previous testing epochs would have resulted in a better comparison for this purpose, though these data are included regardless.

Classification of the crust’s stiffness is possible through means of $V_s$, as suggested by Wentz et al. (2015). Where values of $V_{s,\text{avg}}$ are less than 130 m/s, the stiffness is considered to be low. Where $V_{s,\text{avg}}$ is greater than or equal to 130 m/s and less than or equal to 160 m/s, the crust stiffness is moderate. A high crust stiffness is classified when $V_{s,\text{avg}}$ is greater than 160 m/s.

Pre-injection, the crust stiffness of the site was moderate, with an average $V_s$ of 152.2 m/s between 1 and 6 m. Post-injection, both in 2016 and 2019, a high crust stiffness within the treated zone was recorded, with the average $V_s$ being 206.2 m/s and 186.5 m/s, respectively.

Compression wave velocity has been used to indicate the degree of saturation of a soil. Where the p-wave velocity is over 1,500 m/s, the soil is assumed to be fully saturated (Yang et al. 2004). As shown in Figure 5.2, two sets of cross-hole testing were performed to obtain a $V_p$ profile within the IP. Figure 5.5(a) and (b) show the results of these tests.

At one location, just northwest of the center of the panel, the compression wave velocity decreased from above 1,500 m/s prior to the treatment to well below 1,500 m/s after the resin injection treatment from 2.5 to 6 m. This indicates that the treatment had some effect on the degree of saturation of the soil in the treated zone. At the other location, just southeast of the panel’s center, no noticeable change in p-wave velocity was noted. It is unclear what may have caused this to occur.
Figure 5.5: (a, b) Two compression wave ($V_p$) velocity profiles at the test site, (c) shear wave velocity ($V_s$) before and after treatment, and (d) $V_s$ over time.

However, the other two sites incorporated as part of the RZT also displayed similar desaturation trends to the first set of cross-hole tests at this study’s test site. At Site 4 of the RZT, partial desaturation appeared to occur from 4 to 6 m, and at Site 6 of the RZT from 2 to 6 m (Traylen 2017). By decreasing the level of saturation from fully saturated to partially saturated, the factor of safety against liquefaction increases (Hossain 2010; Yang et al. 2004).
Normalized $V_s$ versus $r_u$ data from Kinney (2018) is shown in Figure 5.6 along with data from the blast testing in Christchurch, New Zealand from 2019. The normalization process decreased the scatter in the datasets and minimizes the effects of density and soil structure degradation on the test results and was accounted for in the additional data from this study. For low values of excess pore pressure ratio, the new data fits well. As an increase in pore water pressure occurs, however, there is larger variation from the averaged trendline. The frequency of readings immediately post-blast was low, leading to a small number of readings with elevated blast-induced pore pressures, making it impossible to identify any trends in the data.

During any seismic event, there is bound to be variation in the shear modulus, as it is dependent on the shear strain that is developed. To compare data across different studies, it is customary to divide the shear modulus by the maximum shear modulus. This $G/G_o$ ratio can then be plotted against the excess pore pressure ratio, $r_u$, to identify the change in behavior of the modulus as pore pressures build up and potentially trigger liquefaction. Figure 5.7 shows the plotted $G/G_o$ data from the testing performed for this thesis in addition to other data as discussed by Kinney (2018), where:

$$G/G_o = \frac{(V_o)^2}{(V_{sf})^2}$$

Equation 5-1

and $V_{sf}$ is the final shear wave velocity. For lower excess pore pressure ratios ($r_u < 0.4$), the data fits the empirical curve well. However, at larger excess pore pressure ratios, the data becomes scattered. It is uncertain what caused this large variation in modulus ratio, though the recording of the $V_s$ data immediately following the blast could have contributed to it. This is because the pore pressures were elevated only for a short amount of time, leading to fewer readings where $r_u > 0.5$ due to the time it took between readings. As relatively few data points were obtained for periods
when $r_u > 0.5$, the results are somewhat inconclusive. Having the capability to rapidly obtain $V_s$ values while the soil is liquefied would be beneficial to future research.

![Figure 5.6](image1.png)

**Figure 5.6**: Normalized shear wave velocity vs. excess pore pressure ratio from blast liquefaction tests.

![Figure 5.7](image2.png)

**Figure 5.7**: Normalized shear modulus vs. excess pore pressure ratio from blast liquefaction tests in comparison with theoretical curve proposed by Kramer and Greenfield (2017) and empirical data based on data from Kinney (2018).

### 5.2.3 Dilatometer constrained modulus (M) and horizontal stress index (K_D)

The constrained modulus, $M$, and the horizontal stress index, $K_D$, are both parameters obtained from the dilatometer test and relate to the stiffness of the soil and horizontal earth
pressures, respectively. Both are used in the calculation of the at-rest earth pressure coefficient, $K_0$, as described in Section 5.2.4.

The modulus is also used in predicting settlements and is inversely proportional to the amount of settlement. As such, an increase in the modulus signifies reduced potential settlement within a profile. Between the pre-injection and post-injection 2016 DMT readings, $M$ increased on average (2 to 6 m depths) approximately 2.3 times the initial values (Figure 5.8(a)), or a 134% increase. In the 2019 reading, the average increase was 140%, showing that the resin injection treatment is still effective with the passing of time.

Figure 5.8: Comparisons of average pre- and post-injection (2016) (a) horizontal stress index and (b) constrained modulus to December 2019 SDMT testing at the IP.
The horizontal stress index increased with the treatment process. Based on Figure 5.8(b), it seems that with time the increase in horizontal stress is retained. The confinement provided by the resin as it expands during the initial injection densifies the surrounding soils, leading to a greater horizontal pressure. As $K_D$ is not a parameter that is easily interpreted by direct analysis, $K_o$ was calculated as part of this study and is presented here.

### 5.2.4 In-situ at-rest earth pressure coefficient ($K_o$)

The at-rest earth pressure coefficient, $K_o$, is a parameter used in estimating the lateral at-rest earth pressure, $\sigma'_h$. Obtaining $K_o$ is a difficult task, however, as it is not directly obtainable from field testing. Correlations relating CPT and DMT readings have been developed for estimating $K_o$ in a soil profile as presented by Hossain and Andrus (2016):

$$K_o = 0.72 + 0.456 \log{OCR} + 0.035K_D - 0.194 \log{\frac{q_c}{\sigma'_{vo}}}$$  \hspace{1cm} \text{Equation 5-2}

where $OCR$ is the overconsolidation ratio, $K_D$ is the horizontal stress index obtained from DMT testing, $q_c$ is the cone tip resistance obtained from CPT testing, and $\sigma'_{vo}$ is the initial vertical effective stress. The OCR is obtained from a correlation established by Monaco et al. (2014):

$$OCR = 0.0344 \left(\frac{M}{q_t}\right)^2 - 0.4174 \left(\frac{M}{q_t}\right) + 2.2914$$  \hspace{1cm} \text{Equation 5-3}

where $M$ is the constrained modulus obtained from DMT testing and $q_t$ is the corrected cone tip resistance recorded from CPT testing. Equation 5-3 is only valid for soils where the DMT material index, $I_D$, is greater than or equal to 1.2 and the CPT soil behavior type index, $I_c$, is less than or equal to 2.6. These constraints ensure the correlation is used for sandy soils as it was intended.

To quantify the improvement in $K_o$ resulting from the resin injection treatment, CPT and DMT tests were performed prior to and following the treatment process, as noted in Table 4.1.
Establishing a base value for $K_o$ for the untreated soil was accomplished by using data from CPT and DMT tests just before the improvement process. Because DMT readings were taken at 0.2 m increments, the $q_c$ and $q_t$ values used in Equation 5-2 and Equation 5-3 were averaged over a 0.2-m range of depth. Multiple CPT readings were taken to identify any variation within the soil profile, so $K_o$ was calculated for each CPT-DMT pair, with the resulting $K_o$ values being averaged together to calculate the unimproved $K_o$. A similar process was followed for the post-improved $K_o$ values, as multiple CPT readings were taken following the improvement process.

A comparison of $K_o$ pre- and post-improvement is shown in Figure 5.9(a). Within the treated layer of 1 to 6 m below grade, 67% of the 0.2-m sublayers (where data were available for both cases) experienced an increase in $K_o$, with an average increase of 36% when compared to pre-improved data. The average pre-improved $K_o$ in the zone of treatment was 0.61, increasing to 0.73 post-improvement. The standard deviation and coefficient of variation (COV) at each depth for the pre- and post-improved values are shown in Table 5.1, as values of $K_o$ were averaged due to the multiple pairs of CPT-DMT data used in the analysis. Low variance in the data shows that the results of the multiple CPT-DMT pairings were all similar with few datapoints having a larger variance because of the changing location of the silty sand bands within the profile. The cone tip resistance in these bands significantly decreases compared to the resistance in the sand, causing the ratio of $M/q_t$ to increase, resulting in a larger calculated OCR and $K_o$. The opposite happened as well, where all the CPT readings except one would have a silt band at the same depth, meaning the ratio of $M/q_t$ would be larger for the reading in the sand layer, resulting in a lower $K_o$ than the other readings at the same depth.
Figure 5.9: Comparison of $K_o$ between the pre-improved IP and (a) post-improvement IP a month after treatment, (b) post-improvement IP three years after treatment, and (c) NP.

Figure 5.9(b) shows the same $K_o$ data for the pre-improved IP compared to the post-improved IP a little over 3 years later. An average increase of 56% in $K_o$ in sublayers where improvement was observed, which occurred in 86% of the treatment zone. The lasting increase in $K_o$ years after initial treatment shows that the resin injection ground improvement is effective at providing confinement within the soil profile over time.

For the blast test a natural area of soil was used as a control to compare against the IP. The $K_o$ for the NP is shown in Figure 5.9(c) along with the untreated IP. There is no clear distinction as to one having a higher average $K_o$ than the other, a good sign that the two soil profiles are similar.
Table 5.1: Standard deviation and coefficient of variation of $K_o$ for plot (a) in Figure 5.9

<table>
<thead>
<tr>
<th>$Z$ [m]</th>
<th>Pre-improved IP</th>
<th>Post-improved IP</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\sigma$</td>
<td>COV [%]</td>
</tr>
<tr>
<td>1.0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.4</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.8</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2.0</td>
<td>0.03</td>
<td>4.9</td>
</tr>
<tr>
<td>2.2</td>
<td>0.03</td>
<td>5.5</td>
</tr>
<tr>
<td>2.4</td>
<td>0.00</td>
<td>1.1</td>
</tr>
<tr>
<td>2.6</td>
<td>0.16</td>
<td>19.7</td>
</tr>
<tr>
<td>2.8</td>
<td>0.04</td>
<td>6.5</td>
</tr>
<tr>
<td>3.0</td>
<td>0.20</td>
<td>27.2</td>
</tr>
<tr>
<td>3.2</td>
<td>0.10</td>
<td>14.4</td>
</tr>
<tr>
<td>3.4</td>
<td>0.00</td>
<td>0.7</td>
</tr>
<tr>
<td>3.6</td>
<td>0.00</td>
<td>0.6</td>
</tr>
<tr>
<td>3.8</td>
<td>0.02</td>
<td>2.5</td>
</tr>
<tr>
<td>4.0</td>
<td>0.00</td>
<td>0.5</td>
</tr>
<tr>
<td>4.2</td>
<td>0.00</td>
<td>0.7</td>
</tr>
<tr>
<td>4.4</td>
<td>0.01</td>
<td>1.2</td>
</tr>
<tr>
<td>4.6</td>
<td>0.03</td>
<td>4.1</td>
</tr>
<tr>
<td>4.8</td>
<td>0.14</td>
<td>18.8</td>
</tr>
<tr>
<td>5.0</td>
<td>0.08</td>
<td>13.6</td>
</tr>
<tr>
<td>5.2</td>
<td>0.04</td>
<td>8.2</td>
</tr>
<tr>
<td>5.4</td>
<td>0.04</td>
<td>7.6</td>
</tr>
<tr>
<td>5.6</td>
<td>0.02</td>
<td>3.6</td>
</tr>
<tr>
<td>5.8</td>
<td>0.01</td>
<td>1.0</td>
</tr>
<tr>
<td>6.0</td>
<td>0.20</td>
<td>28.1</td>
</tr>
<tr>
<td>6.2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>6.4</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>6.6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>6.8</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7.0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7.2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7.4</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7.6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7.8</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>8.0</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
5.2.5 Persistence of treatment

Understanding the persistence of the resin injection treatment over time is important. By comparing post-treatment data obtained soon after the ground improvement process occurred to pre-treatment data, an initial improvement in soil parameters can be calculated. Since the treatment process occurred in 2016 and the blast testing occurred in 2019, an additional dataset was collected to compare against the pre-treatment data. A summary of the improvement due to the resin injection ground improvement process is presented in Table 5.2.

Table 5.2: Increases in soil parameters within the treated zone

<table>
<thead>
<tr>
<th>Test</th>
<th>$q_c$</th>
<th>$q_{clnCs}$</th>
<th>$V_s$</th>
<th>$M_{DMT}$</th>
<th>$K_o$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2016 post-inj</td>
<td>105%</td>
<td>89%</td>
<td>25%</td>
<td>44%</td>
<td>35%</td>
</tr>
<tr>
<td>2019</td>
<td>100%</td>
<td>90%</td>
<td>17%</td>
<td>28%</td>
<td>23%</td>
</tr>
</tbody>
</table>

For each of the parameters discussed in section 5.2 an increase was still present three years after the treatment occurred. For shear wave velocity, a slight decrease from the initial post-treatment values was observed, with each of the other parameters remaining nearly the same. These data show that the efficacy of the resin injection treatment holds with time and has little to no degradation in the improvement.
6 SITE INSTRUMENTATION AND BLAST TEST PROCEDURE

6.1 Explosives setup and blast procedure

Blast densification of sand has been used as a method for improving loose, saturated cohesionless soils (Byom 1963; Ivanov 1972) for over 60 years. More recently, it has been used as a method of liquefying soils to test the efficacy of different ground improvement methods, both shallow and deep (Amoroso et al. 2017; Amoroso et al. 2020; Ashford et al. 2000; Ashford et al. 2004; Charlie et al. 1988; Gohl et al. 2001; Ishimwe 2018; Rollins et al. 2021; Rollins et al. 2001; Wentz et al. 2015).

These full-scale tests are extremely useful because researchers can safely simulate the effects of an earthquake without having to wait for one to occur naturally. Although the blast testing cannot exactly replicate ground motions that are associated with earthquakes, the effects of the blasting such as liquefaction and the subsequent settlements can be evaluated (Gianella and Stuedlein 2017; Gohl et al. 2001; Wentz et al. 2015).

Ashford et al. (2004) noted that during blast-induced liquefaction the porewater pressure is almost instantaneously increased by compression waves generated from the blasting, while in earthquake-induced liquefaction the porewater pressure slowly increases by the shear waves propagating up through the soil. Stuedlein et al. (2021) reiterates how primary $p$-waves induce the instantaneous and ephemeral spikes in porewater pressures, while the $s$-waves are the main cause of generated excess pore pressures that persist following each individual blast.
To better simulate the build-up of excess pore pressure, more recent blast-induced liquefaction tests have detonated charges with a delay of 800 to 1000 milliseconds between charges. Despite the differences between blast- and earthquake-induced pore pressures, the velocity and strain levels resulting from blasting were found to be comparable to that of an actual earthquake. Based on the success of previous test regimes, it was determined by researchers from BYU that blast testing would be useful in evaluating the efficacy of the resin injection ground approach. Rollins et al. (2021) found that settlement from blasting can be very similar to that predicted for an earthquake.

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 late September and early October of 2016, as discussed in Chapter 5. 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 meters. Each blast hole (10 m deep) was cased with a 75 mm diameter PVC pipe throughout the depth of the hole to prevent caving and to allow for dewatering prior to installing the explosive charges.

Two decks of explosives were placed in each pipe: one centered at a depth of 4 m and one centered at a depth of 8.5 m. The explosive charges were 0.6 to 1.0 m long. The charge weight of each explosive in the lower deck was 2.4 kg while each explosive in the upper deck had a charge weight of 1.2 kg, as shown in Figure 6.1. 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. To ensure safety, each pipe was capped off and labelled prior to blasting and then loaded with a surcharge of gravel bags to avoid expulsion of ejecta (see Figure 6.2).

![Capped PVC pipe where charges were lowered to the appropriate depths.](image)

**Figure 6.1:** Capped PVC pipe where charges were lowered to the appropriate depths.

In each blast test, the lower deck of explosives was first set off sequentially according to the blast hole numbers indicated in Figure 6.3 with detonations alternating from opposite sides of the panel to simulate the shearing induced during an earthquake. Afterwards, the upper deck of explosives was detonated with the same sequence. The delay between detonations was 600 milliseconds. Both panels had the same sequence of blasting as shown in Figure 6.3.

The blast tests for the two panels were conducted separately (i.e., blast 1 in the IP and blast 2 in the NP) to limit the effects of superposition and to simplify the comparison of the effects of the blast induced liquefaction on the IP and the NP, separately. The first blast took place on December 9, 2019, and the subsequent blast took place the following day to allow time for post-blast surveys and observations to be made.
Throughout this report, the perimeter of the blast holes is referred to as the blast circle. Most liquefaction and settlement that took place occurred within these blast circles. It was anticipated that the blast pressures from each individual detonation would be greatest near the blast holes but that the combined cyclic stresses at the center of the blast ring would be the largest due to the surrounding blast holes. Further from the center of the panels and blast holes, it was expected that the pressures generated by the blasts would dissipate more rapidly than in the center.
Figure 6.3: Blast hole configuration in the IP with respect to the injection points.
6.2 Pore pressure instrumentation layout

Piezoresistive piezometers, commonly referred to as pore pressure transducers (PPT) throughout this report, were installed from 3 to 8 m at each meter of depth and at 10 m. However, the PPTs installed at a depth of 6 m stopped recording accurate data shortly after pushing them into the ground, most likely due to the instruments encountering fallen gravel at the bottom of the holes. Due to this mishap, the data for 6 m have been excluded.

The PPTs, which recorded readings at a frequency of 20 Hz, were used to measure the generation and subsequent dissipation of excess pore water pressures induced by the blasts. Seven PPTs were installed in each test panel, around the center of the panel, on a 1 m radius circle. As shown in Figure 6.5, the profilometer in the NP was installed where the 3 m PPT was planned to be located, so the PPT was installed in the center of the panel instead. An additional PPT was installed equidistant from the center of the IP and NP at a depth of 4 m adjacent to an array of four accelerometers. The data from this PPT can be used to understand the non-linear behavior of the soil due to liquefaction, coupling the shear strains gathered by the accelerometer array with the excess pore pressures, as well as to observe pore pressures at a distance from the blast location.

Pore pressure data were critical to obtain because the calculation of the excess pore pressure ratio \((r_u)\) can help determine if the observed soil layer has liquefied. The preparation and installation process of the PPTs followed the procedure for piezoresistive piezometers given by Rollins et al. (2005) and is summarized as follows.

The holes in the tips of the protective cones were stuffed with clean cotton to avoid clogging during installation and blasting. They were then boiled in water to ensure a fully saturated condition and to remove any air bubbles, as these would create issues during testing if not addressed. Under saturated conditions, the piezometers were then screwed into the cone tips and
inserted into the body of the cone to ensure the safety of the instrumentation during the pushing process. Once tightened together, the assemblage was tapped repeatedly to remove any remaining air bubbles. Before removing it from the deaired water, a rubber membrane was placed around the cone to keep the tip saturated and prevent contamination during the installation process prior to being pushed into the native soil.

Before installing the PPTs, holes were drilled at the designated locations to a depth 0.3 m above the desired depth of the sensor. A bentonite slurry was then pumped into each hole to prevent wall collapse and allow for easy access to the native soil. The PPT assemblage was then attached to a push rod, similar to how a cone penetrometer works, and lowered to the bottom of the hole. The instrument was then pushed the remaining 0.3 m into native soil, which served two purposes: breaking the rubber membrane to allow for the piezometer to receive an accurate reading and to make the PPT be completely surrounded by native soil in a fully saturated condition.

Each PPT was connected to the same laptop computer located in a safe area away from the blast circles via electrical wiring. As noted previously, pore pressures from each transducer were recorded at 0.05 second intervals. Data were recorded prior to and following each blast for a period of time to establish a baseline water pressure and to observe the transient noise within the data. These readings were necessary for accurately calculating the change in porewater pressure from the static condition. Additionally, a metal wire was attached to each cone casing to assist in the process of removing the PPTs from the ground following the blast testing. A schematic of this assemblage is shown in Figure 6.4.
The depths for the PPTs were selected to gather pore pressure data at discrete locations throughout the potentially liquefiable layer. Unit weight measurements gathered from the flat dilatometer tests were used to determine the following vertical effective stresses at each depth where PPTs were installed. The data gathered from the SDMTs are shown in Table 6.1 along with the soil type based on the material index, I_D. Most transducers were in soils comprised of silty sands, though there appears to be a layer of cleaner sand around 4 to 5 m deep. Figure 6.5 shows a plan view of the PPT locations in the NP and IP, as well as the PPT located between both panels.
Figure 6.5: Plan view of pore pressure transducer layout (with depths) across the test site.
Table 6.1: Initial vertical effective stress readings at each pore pressure transducer used in the computation of $r_u$ during blast activities

<table>
<thead>
<tr>
<th>Natural Panel</th>
<th>Vertical Effective Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td>Soil Type</td>
</tr>
<tr>
<td>3</td>
<td>Silty Sand</td>
</tr>
<tr>
<td>4</td>
<td>Silty Sand</td>
</tr>
<tr>
<td>5</td>
<td>Sand</td>
</tr>
<tr>
<td>7</td>
<td>Silty Sand</td>
</tr>
<tr>
<td>8</td>
<td>Silty Sand</td>
</tr>
<tr>
<td>10</td>
<td>Silty Sand</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Improved Panel</th>
<th>Vertical Effective Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td>Soil Type</td>
</tr>
<tr>
<td>3</td>
<td>Silty Sand</td>
</tr>
<tr>
<td>4</td>
<td>Sand</td>
</tr>
<tr>
<td>5</td>
<td>Silty Sand</td>
</tr>
<tr>
<td>7</td>
<td>Silty Sand</td>
</tr>
<tr>
<td>8</td>
<td>Silty Sand</td>
</tr>
<tr>
<td>10</td>
<td>Silty Sand</td>
</tr>
</tbody>
</table>

6.3 Settlement instrumentation and layout

Integral to the study was the need to obtain accurate measurements of settlements induced by both blast sequences. Different systems were set in place to measure ground surface settlements, settlement versus depth, and surface settlements versus time. Each set of measurements and the instrumentation required to obtain the data will be discussed further.

6.3.1 Ground surface settlement

Ground surface settlements were measured across the test site using fifty-three fixed elevation indicators, or wooden survey stakes. These were placed in a straight line from the center of the improved panel to the center of the natural panel, extending approximately 8 m past the
outer blast ring of each panel, and were manually surveyed at the site. The stakes were spaced at 1-m intervals across the whole site, where possible.

During each blast test concrete blocks (60 cm wide x 120 cm long x 60 cm high), or kentledge, were placed atop the panel to simulate a foundation load of around 14 kPa. As such, the wooden stakes were removed during that time and a mark was painted on top of the concrete kentledge to be surveyed instead. Following the first blast at the IP, the survey stakes were reinstalled to take pre- and post-blast settlement measurements for the second blast at the NP. The survey stakes near the center of the site were placed slightly askew due to the placement of the accelerometers and PPT.

An automatic survey level and a barcode-style survey rod were used to measure the elevation of each survey point. The accuracy of the level was 0.0305 cm (0.001 ft). Each survey stake and paint mark were surveyed prior to blasting and approximately one hour following each blast. This allowed for the incremental settlement due to each blast to be calculated as part of this study. Prior to taking the survey measurements, a back-sight point located 70 m away from the test site was measured to ensure that the elevation of the tripod did not change between surveys. It was assumed that data points which oscillated around the origin by less than 2 mm to be zero to simplify the settlement profile and to account for minor movements in the survey stakes from the blast shocks.

6.3.2 Settlement vs. depth

The arrangement of the blast charges was such that the maximum imparted energy would be felt in the center of the respective blast ring. Correspondingly, it was anticipated that this would be the point of maximum settlement within each panel. Therefore, in each panel a profilometer was installed near the center of the panel. In the NP, the location of the profilometer and 3 m PPT
were switched due to issues the drilling team encountered when attempting to install the profilometer. The profilometer consists of a corrugated PVC drainage pipe fitted with steel cable ties at 0.5 m intervals along the length of the pipe. This pipe, as shown in Figure 6.6, was inserted into a drilled hole and grouted at the bottom to remain in place despite large buoyant forces. A PVC access pipe was then inserted coaxially into the corrugated pipe to allow for measurements to be taken.

![Figure 6.6: (Left) Profile view of profilometer; (Right) Picture of metal rings attached to corrugated pipe for settlement readings.](image)

After installation, the saturated sands naturally collapsed around the lower part of the corrugated pipe. The top of the annulus between the corrugated pipe and the drilled hole stayed open, however, which facilitated the manual backfilling and consolidation of the annulus using surficial soil that had been ejected during the drilling process. It was imperative to have the exterior pipe be in contact with the soil to record accurate settlement data, as the settlement of the soil would not be reflected in the profilometer if it was not in direct contact with the soil.

During blasting and subsequent reconsolidation of the soil surround the corrugated pipe, the pipe can shorten or elongate with the strain of the surrounding soil. The interior PVC pipe assisted in preventing horizontal displacement of the profilometer and allowed access for the
Sondex Settlement System to be used. The Sondex probe was lowered through the profilometer, detecting the presence of the metal rings as it went down. Elevations of each ring were taken prior to and after blasting, thus reflecting the settlement or heave in the soil profile due to the blast-induced liquefaction by comparing the post-blast depth of each ring to its initial depth. The profilometers were labeled NP_Prof for the NP and IP_Prof for the IP. Each profilometer was fitted with 30 rings and embedded to a depth of approximately 15 m.

6.3.3 Surface settlement following blast sequences

Ground surface settlements were also monitored during the blast sequences and immediately after. These observations were made to provide a correlation between the rate of ground surface settlement and the rate of dissipating porewater pressures in the profile. These data were obtained by placing two grey PVC pipes orthogonal to the ground surface within each test panel as shown in Figure 6.7 below.

Figure 6.7: Oblique view of the four grey PVC surface settlement posts and the wooden surface settlement stakes across the test site (IP in the foreground).
Each survey post was embedded approximately 1 m into the ground via drilled holes, as the layer of gravel remaining from the resin injection process made it extremely difficult to excavate by hand. Each panel had two survey posts; one was approximately 1 m from the center of the panel and the other was approximately 2 m from the center of the panel. Once inserted into the ground, surficial soil from the drilling process, as well as some pea gravel, was lightly tamped to secure the posts from falling over during the blasting sequence and to avoid disturbance of the native soil. Bar codes were then duct taped to each pipe to be read by an automatic survey level before, during, and after the blasting sequences. The tripod for the auto-level was set up approximately 40 m away from the center of the panel being blasted to avoid the effects of blast-induced settlement. This distance was also used as a buffer to ensure the safety of researchers during the blast, per the requirements of the blast engineers.

Similar to the readings for the wooden survey stakes, the accuracy of the automatic survey level was 0.0305 cm (0.001 ft). Before and after each set of measurements was taken, a back-sight elevation was recorded to adjust for any movement of the ground surface where the tripod was located. The four grey PVC surface settlement posts were labeled as shown in Table 6.2 below. The layout of the settlement instruments, described in this chapter, were recorded using a high-resolution GPS unit by Dr. Michael Olsen of Oregon State University and can be seen in Figure 6.8 with respect to the blast rings of the NP and the IP.
Researchers from Oregon State University used ground-based lidar to measure surface movements induced by the blasting sequences and resulting effects of liquefaction. Surveys were conducted prior to and following each blast sequence using a Leica GS18 real-time kinematic (RTK) rover coupled with a GNSS base station set on a 2 m fixed height bipod. For each epoch, set survey control points were placed away from the blast rings to ensure no settlement would be encountered. Global positioning and orientation of the control points were located and subsequently, targets in the test site were surveyed to allow for the relative positioning of other objects of interest.

Five 360° scans using a Leica P40 laser scanner were performed per survey to capture lidar data needed for constructing models. A Leica GS18 GNSS was mounted on top of the laser scanner to help derive initial estimations of the scan origin.

<table>
<thead>
<tr>
<th>Marker</th>
<th>Distance from Center of Natural Panel ft (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>3.28 (1.0)</td>
</tr>
<tr>
<td>P2</td>
<td>6.56 (2.0)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Marker</th>
<th>Distance from Center of Improved Panel ft (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P3</td>
<td>3.28 (1.0)</td>
</tr>
<tr>
<td>P4</td>
<td>6.56 (2.0)</td>
</tr>
</tbody>
</table>
Figure 6.8: Locations of the surface survey stakes and posts across the test site.
Figure 6.9: Locations of the blast holes, resin injection points, pore pressure transducers, accelerometers, and survey equipment for settlement measurements.
Once gathered, the GNSS data were used to derive an initial coordinate for the base station and survey control points using the Leica Infinity software. These points, along with measurements from the total station, were used to derive global coordinates using STAR*NET software. Data from the Leica P40 scans were then processed in Leica Cyclone to register each scan into a global coordinate system and to generate point clouds. Following this, a ground filtering algorithm was used to remove all non-ground points, leaving a point cloud solely comprised of ground points. A digital terrain model (DTM) was then created for each survey epoch: IP pre-blast, IP post-blast, NP pre-blast, and NP post-blast. Differences between each epoch were calculated using a change detection algorithm to measure any settlement across the test site.

At the same time as the ground-based lidar was being conducted, aerial and terrestrial based structure from motion (SfM) surveys took place. These were carried out by the Brigham Young University Research in Optimized Areal Modeling (ROAM) group, directed by Dr. Kevin Franke, using an Unmanned Aerial Vehicle (UAV) equipped with a Zenmuse X4S camera, flying 25 m above the ground taking pictures as it flew in a grid pattern over the test site. Supplemental low-elevation-flight photographs were taken with the same UAV to assist in the generation of the model. Ground SfM imagery was captured using a Nikon D750. Four photogrammetric models were created in total: IP pre-blast, IP post-blast, NP pre-blast, and NP post-blast. Each model was created and processed using approximately 1,700 photos in Bentley Context Capture (version 15) as a point cloud and then converted to a textured mesh for viewability.
6.4 Additional instrumentation

Accelerations were measured by researchers from the National Institute of Geophysics and Volcanology (INGV) using eight in-hole triaxial accelerometers, or geophones (DA01-DA04 and AA1-AA4). Digital accelerometers DA01-DA04 were installed about the center of the NP using a 1 m-square configuration. Analog accelerometers AA1-AA4 were installed approximately midway between the two panels in the same square configuration. Both sets of accelerometers were installed in pre-drilled holes, filled with bentonite slurry, at a depth of 4 m. Data collected from these allows for the estimation of blast-induced shear strains and stresses.
The complete pore pressure and surface settlement instrumentation plan view layout across the test site is shown in Figure 6.9, with a typical subsurface cross section at the improved panel shown in Figure 6.10.
7 RESULTS FROM BLAST 1 AROUND IMPROVED PANEL (IP) AND BLAST 2 AROUND NATURAL PANEL (NP)

The pore pressure responses and resulting settlement in the IP and the NP from the two blast sequences are reviewed, compared, and contrasted in the subsequent sections.

7.1 Excess pore pressure measurements

Blast 1, centered around the improved panel, occurred at 12:05 on December 9, 2019. Excess pore pressures were measured by the PPTs as described previously in section 6.2. The excess pore pressure ratios at each PPT were computed using Equation 2-1. The vertical effective stresses used in calculating the excess pore pressure ratios were derived using the unit weights from the SDMT as discussed in section 4.1 and are shown in Table 6.1. An $ru$ of 1.0 signifies that the excess pore water pressure is equal to the confining pressure of the soil. With these two forces in equilibrium, liquefaction occurs. However, because of the uncertainty in factors such as the soil’s precise unit weight and existing hydrostatic pressures, an excess pore pressure ratio of 0.90 or higher suggests that the soil is essentially liquefied, with excess pore pressure ratios of 0.80 or higher suggesting incipient liquefaction.

The excess pore pressures ratio during blast 1 in the improved panel (IP) are plotted with time in Figure 7.1. The inset in Figure 7.1 is a snapshot of the excess pore pressure ratios immediately following the initial detonation in the first blast sequence. The time is relative to the
initial detonation of each blast sequence, which occurs at 0 seconds. Piezometers gathered data for approximately 20 minutes following the blast sequence.

As blast detonations occurred, transient spikes in the excess pore pressure ratios were recorded with values higher than 1.0. These spikes represent large and ephemeral water pressure changes (plus and minus) due to the compression (P) waves created by the blast arriving at the PPTs. Because the transient spikes are relatively unimportant relative to the residual pore pressures following the passage of the P-waves, the raw pore pressure measurements were smoothed to remove a majority of the transient pulses. This was done by using a 50-point moving average, which also allowed for better representation of the residual pore pressures following the blasts, as the resting pore pressure measurements tended to fluctuate due to background “noise” in the sensors.

As shown in Figure 7.1, excess pore pressure ratios of 0.8 and above lasted between 12 to 50 seconds depending on the depth, with the shallower layers remaining in a liquefied state longer than the deeper layers. Dissipation of pore pressures occurred more slowly at shallower depths, presumably due to the upward flow of water from the underlying layers. As the porewater must resist the downward force of the soil for liquefaction to occur, it is natural to presume that excess pore pressures at greater depths will dissipate faster without the continuation of forces generated by the blasting. Though this is the case, pore pressures at depth are still much greater than that at shallower depths. While the excess pore pressure ratios may be comparable throughout the profile immediately following the blast, excess pore pressures are greater at deeper elevations than at shallower depths. Excess pore pressure ratios at 8 and 10 m reached 0.1 in just 4 minutes following the blasts, while excess pore pressure ratios at 3, 4, 5, and 7 m took around 20, 17, 13, and 9 minutes, respectively.
The $r_u$ values recorded in the NP during blast 1 are shown in Figure 7.2. A maximum excess pore pressure ratio of 0.13 occurred, at a depth of 5 m, signifying that pore pressures generated in the natural panel did not come close to the levels required for liquefaction to occur. By the next day when the second blast test was conducted, the pore pressures had returned to their original hydrostatic values.

Blast 2, centered around the natural panel, occurred at 15:00 on December 10, 2019. The pore pressure data from this blast were also smoothed using a 50-point moving average to reduce transient spikes in the pore pressure time histories and to eliminate the background “noise” recorded by the PPTs. Peak excess pore pressure ratios from blast 1 in the IP are presented in Figure 7.3, and peak $r_u$ values from blast 2 in the NP in Figure 7.4. For both blasts, liquefaction occurred in the panel where the blast charges were detonated, as excess pore pressures exceeded the threshold for incipient liquefaction.

Figure 7.5 below shows the excess pore pressure ratios versus time for the NP during blast 2. The time is relative to the beginning of blast 2, occurring at 0 seconds. Piezometers continued to record data for approximately 20 minutes following the blast sequence. Except for the excess pore pressure at 3 m, all excess pore pressures had returned to 0.1 or below by 10 minutes after the blasts. At the end of the recorded data for the 3 m PPT, the excess pore water pressure was just over 0.1. Excess pore pressure ratios above 0.8 lasted between 14 to 42 seconds.
Figure 7.1: Residual excess pore pressure ratio in the IP during blast 1 in the IP at 3, 4, 5, 7, 8, and 10 m depths. Average peak residual pore pressure ratio with depth during blast sequence (shown inset).
Figure 7.2: Residual excess pore pressure ratio in the NP during blast 1 in the IP at 3, 4, 5, 7, 8, and 10 m depths.
Figure 7.6 shows the excess pore pressure ratio versus time in the IP during blast 2 in the NP. A maximum excess pore pressure ratio of 0.19 was measured at a depth of 7 m. Similar to the data shown in Figure 7.2, elevated pore pressures induced by blasting in the opposing test panel were not sufficient to produce liquefaction. Though blast testing in the IP occurred the day before, pore water pressures nearly returned to their initial hydrostatic state by the time of blast 2 the following day. Slight variation in the resting hydrostatic pressures were noted between the static readings prior to blasts 1 and 2, although these are most likely attributed to the vertical displacement of the piezometers subsequent to the detonated charges.

Figure 7.3: Comparison of peak excess pore pressure ratio, $r_u$, measured during blast 1 in the IP and in the NP.
Figure 7.4: Comparison of peak excess pore pressure ratio, $r_u$, measured during blast 2 in the NP and in the IP.

A comparison of peak excess pore pressure ratios is shown in Figure 7.7. No significant difference in excess pore pressure ratios is discernable, though it appears that the values in the IP following blast 2 in the NP are slightly greater than the values in the NP following blast 1 in the IP. This could potentially be attributed to the fact that the soils in the IP had been liquefied during the first blast sequence the day before, thus reducing the residual strengths of the soils in the IP leading to slightly higher pore pressures than what would have occurred if the order of the blast tests had been reversed.

In addition to piezometric data obtained from pore pressure transducers in both panels, a seismic dilatometer was installed in the natural panel approximately 3 m from the center of the piezometer array, at a depth of 4 m, to record data during blast 2.
Figure 7.5: Residual excess pore pressure ratio in the NP during blast 2 in the NP at 3, 4, 5, 7, and 10 m depths. Average peak residual pore pressure ratio with depth during blast sequence (shown inset).
Figure 7.6: Residual excess pore pressure ratio in the IP during blast 2 in the NP at 3, 4, 5, 7, 8, and 10 m depths.
Excess pore pressure ratio versus time curves obtained from PPT_4m_NP are plotted along with those measured by the SDMT data in Figure 7.8. Both sets of data show extremely similar trends in the excess pore pressure ratio following the blast, with pore pressures dissipating at the same rate as measured by both sets of instruments. This verification of the $r_u$ time histories through means of PPT and SDMT indicate that both methods are viable for obtaining measurements necessary to indicate the occurrence of liquefaction. However, the readings of the SDMT less frequent than data recorded by the PPTs, so interpolation must be made between adjacent readings to create a best-fit line, while the output of the PPTs provides a smoother and more continuous record of data. As the first datapoint was not recorded until 48 seconds following the initial detonation due to safety concerns immediately following the blasting sequence, the largest $r_u$ as recorded by the SDMT was 0.72.

Figure 7.7: Comparison of peak excess pore pressure ratio, $r_u$, measured during blast 1 in the IP and blast 2 in the NP.
As the ultimate peak excess pore pressure ratios recorded immediately following the blasting sequences are not as critical as the average peak residual \( r_u \) perpetuated by the shear waves, the duration of time where \( r_u \) is greater than 0.8 is most crucial for identification of incipient liquefaction within the soil mass. Figure 7.9 compares the average peak residual excess pore pressure ratio between the NP and the IP following the blast sequences in the respective panels over time. With both blasts, pore pressures tended to dissipate faster at greater depths. The outlier to this pattern tended to be the pore pressures recorded at 7 m. This is most likely to have occurred because piezometers at that depth were located (vertically) between the blast charges which were placed at 4 and 8.5 m, creating the largest increase in pore pressures between those depths. Additionally, the dissipation of pore pressures at this depth would be slowed by the need for excess pore pressures above or below to begin dissipating prior to drainage.
Another trend observed was that pore pressures tended to dissipate faster in the natural panel than in the improved panel. Densification of the soil surrounding the injection points and the decreased permeability of the soil due to the presence of the polyurethane resin are factors that contributed to this effect. While extended periods of elevated pore pressures are not desirable, the total length of time in which the average excess pore water pressure ratio exceeded 0.8 did not differ drastically between the two panels. In both blast tests, liquefaction is thought to have occurred in the range of depths that were instrumented.

![Figure 7.9: Dissipating $r_u$ values with depth between 10 and 240 seconds after initial detonation during blast 1 and blast 2.](image)

Comparing the change in excess pore pressure ratios between depths within the improved panel highlights the effects of partial or full saturation as discussed in section 5.2.2. Figure 5.5(a) shows that between 0 and 6 m the ground is partially saturated, as $V_p$ is less than 1500 m/s, the
approximate compression wave velocity in water. As seen in Figure 7.10, excess pore pressures spike almost instantly when the soil is fully saturated. Conversely, where the soils are partially saturated the buildup of pore pressures is significantly slower. Multiple case studies reviewed by Hossain (2010) indicate that for partially saturated soils the CRR increases, thereby increasing the factor of safety against liquefaction when compared to fully saturated soils.

Figure 7.10: Pore pressure generation in the improved panel at 3 and 7 m depths.

Between panels, pore pressures tended to generate more rapidly in the natural panel than in the improved panel. Comparing excess pore pressure ratios at the same depth in each panel, such as at a depth of 3 m shown in Figure 7.11, this trend is consistent. Additionally, the magnitude of the transient spikes immediately following the detonation of charges are larger in the NP than the IP. These trends hold true for the data recorded at each discrete depth, though there are a few instances where the spikes in the IP are larger due to the distance of the PPT to the blast charge during the blasting sequence.
7.2 Ejecta

Subsequent to each of the blasting sequences, ejecta from the PPT boreholes was observed at each panel. This ejecta mainly consisted of the bentonite slurry that was used during installation of the piezometers to prevent the sidewalls of the boreholes from collapsing. Trace amounts of the liquefiable sands were present at the surface within the slurry, though no characteristic sand boils were observed. It was presumed that significant sand ejecta and surficial heave would be observed at the center of the blast circles where the effects of the blasting would be most noticeable. However, as the concrete kentledge covered a majority of the area within the array of piezometers where these phenomena were expected to occur, the surcharge provided by the concrete kentledge may have prevented these from manifesting. Additionally, no ejecta were observed extruding from the blast holes surrounding the panels, as gravel and sandbags had been placed atop the holes prior to blasting to prevent this from happening.
7.3 Liquefaction-induced settlements

This section will report the ground surface settlements measured in real time during and after the blast, the ground surface settlements obtained by ground-based lidar, and the settlements observed with depth.

7.3.1 Settlement vs. time

Survey measurements were taken using an auto-level coupled with the survey posts that were installed 1 and 2 m from the center of each panel. The posts were surveyed immediately following the blasts as soon as it was deemed safe to exit the blast shelter. Alternating measurements were rapidly taken between the 1 and 2 m survey posts for the first 5 minutes after the blast, then approximately every minute until 50 minutes had passed from the final charge detonation, which is represented by t=0. Figure 7.13 provides plots of both the total settlement and normalized settlement versus time for the NP and IP during blasts 1 and 2, respectively, for 15 minutes after the blast. For the improved panel, 90% of the settlement recorded by the survey posts occurred at 126 seconds and 170 seconds for the posts 1 and 2 m away, respectively. For the natural panel, 90% of the settlement recorded by the survey posts occurred at 135 and 160 seconds.
for the posts 1 and 2 m away, respectively. At the end of the 50 minutes, the total settlements observed were 62.2 and 68.0 mm for the IP, and 61.6 and 70.1 mm for the NP at 1 and 2 m away from the center of the respective panels. Therefore, the settlement versus time curves were quite similar for both the IP and NP.

In the improved panel, settlements recorded 2 m away from the center of the panel averaged 90.2% of the settlements recorded 1 m away. For the natural panel, settlements observed 2 m away from the panel center averaged 86.5% of the settlements 1 m away. Though the variation is minor, the settlement observed in the improved panel was more uniform than the settlements in the natural panel. Despite having nearly the same total settlement, a more uniform settlement would be less detrimental to a superstructure than a larger differential settlement.

For Figure 7.13(c) and (d), the settlement has been normalized by the maximum settlement in each case to facilitate comparisons. After normalization, the settlement curves at 1 and 2 m from the center follow very similar trends with time. Plots of the average excess pore pressure ratio versus time are also plotted in Figure 7.13(c) and (d) to provide context. Settlement occurs in a very non-linear fashion relative to the average \( r_u \). Typically, about 80% of the liquefaction-induced settlement occurs as the average \( r_u \) decreases from 1.0 to 0.6; whereas the remaining 20% of the settlement requires the average \( r_u \) to decrease from 0.6 to 0.
Figure 7.13: (a) Measured ground settlement with time for the IP during blast 1 and (b) for the NP during blast 2. Normalized ground settlement and average $r_u$ for (c) the IP in blast 1 and (d) the NP in blast 2.

7.3.2 Settlement vs. horizontal distance

A superimposed plot of ground surface settlement across the NP and the IP is presented in Figure 7.14. Settlement data presented in this section were obtained from auto-level surveys using the fixed elevation indicators, or wooden survey stakes. Both panels recorded a maximum surface settlement of around 76 to 80 mm, occurring near the center of each panel. Some slight variations in the sections between 25 and 30 m are due to the differential settlement of the concrete kentledge, as it was not viable to install and survey the stakes while the kentledge was present, though a generalized trend curve results in a relatively good approximation. The magnitude of blast-induced
surface settlements decreased moving away from the center of the panels, with negligible settlements occurring at distances greater than 12 m away from the center of the blast array.

Although the maximum ground surface settlement was very similar at the center of both arrays, greater settlement was measured for the IP than for the NP at locations away from the center. This observation suggests that the differential settlement about the maximum settlement would be less for the improved panel than for the natural panel. Additionally, on the edges of the surcharged kentledge region, more differential settlement was observed in the NP than the IP. The surface settlement profile across the test site through the NP and IP is shown in Figure 7.15.

![Figure 7.14: Superimposed ground surface settlements of IP and NP based on measurements with the wooden survey stakes.](image-url)
Figure 7.15: Settlement across the test site on a line between the center of the IP and NP.

7.3.3 Ground-based lidar settlement

As described in Section 6.3.3, ground-based lidar surveys were performed before and after each blast sequence by researchers from Oregon State University. The difference between the surveys was computed to produce a topographic model of the ground settlement across the site. This information is useful because it is not limited to a discrete point or line but is semi-continuous over a large area. Models of ground settlement at the IP and NP were combined to produce Figure 7.16. Similar to the settlement data obtained by the survey stakes, settlements in the general area surrounding the NP and IP are typically in the range of 70 to 80 mm, which is in good agreement with the survey data with the auto-level. However, as the lidar data contains tens of thousands of
Figure 7.16: Overall site settlement based on ground-based lidar data.
points and covers a larger area, more complex analyses can be performed to better understand the overall picture of blast-induced settlement at the site. The settlement contours in Figure 7.16 seem to indicate that the settlement is more uniformly distributed around the IP than in the NP.

Analyzing the settlement within the blast circles of each panel, specifically in the areas where the concrete kentledge was located, was accomplished using zonal statistics within ArcGIS Pro. Polygons were drawn that indicate the general boundaries of the concrete kentledge within each test panel. Additional polygons were drawn that outline each individual concrete block in the blast circle. Using zonal statistics on the lidar data within those boundaries the following statistics were assessed: minimum, maximum, mean, and standard deviation of settlements with each respective polygon. The average settlement for each individual block, whose outlines are shown in Figure 7.17, and for the general area of concrete kentledge, are provided in Table 7.1.
Table 7.1: Average settlement for individual concrete blocks and general kentledge area

<table>
<thead>
<tr>
<th>IP</th>
<th>MIN</th>
<th>MAX</th>
<th>MEAN</th>
<th>STD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-0.2063</td>
<td>0.301</td>
<td>-0.0395</td>
<td>0.1145</td>
</tr>
<tr>
<td>2</td>
<td>-0.2036</td>
<td>0.3416</td>
<td>-0.0734</td>
<td>0.0420</td>
</tr>
<tr>
<td>3</td>
<td>-0.2278</td>
<td>0.2037</td>
<td>-0.0782</td>
<td>0.0262</td>
</tr>
<tr>
<td>4</td>
<td>-0.2133</td>
<td>0.0758</td>
<td>-0.0787</td>
<td>0.0152</td>
</tr>
<tr>
<td>5</td>
<td>0.1156</td>
<td>-0.0584</td>
<td>-0.0801</td>
<td>0.0053</td>
</tr>
<tr>
<td>6</td>
<td>0.1219</td>
<td>0.1156</td>
<td>-0.0785</td>
<td>0.0147</td>
</tr>
<tr>
<td>7</td>
<td>0.2920</td>
<td>0.1464</td>
<td>-0.0793</td>
<td>0.0353</td>
</tr>
<tr>
<td>8</td>
<td>0.1152</td>
<td>-0.0552</td>
<td>-0.0767</td>
<td>0.0060</td>
</tr>
<tr>
<td>9</td>
<td>0.1111</td>
<td>-0.0218</td>
<td>-0.0763</td>
<td>0.0070</td>
</tr>
<tr>
<td>10</td>
<td>0.1152</td>
<td>-0.0620</td>
<td>-0.0803</td>
<td>0.0055</td>
</tr>
<tr>
<td>11</td>
<td>-0.1170</td>
<td>-0.0655</td>
<td>-0.0807</td>
<td>0.0054</td>
</tr>
<tr>
<td>12</td>
<td>-0.1205</td>
<td>0.3256</td>
<td>-0.0687</td>
<td>0.0410</td>
</tr>
<tr>
<td>13</td>
<td>-0.3101</td>
<td>0.0298</td>
<td>-0.0723</td>
<td>0.0231</td>
</tr>
<tr>
<td>14</td>
<td>-0.2050</td>
<td>0.0575</td>
<td>-0.0728</td>
<td>0.0177</td>
</tr>
<tr>
<td>15</td>
<td>-0.1599</td>
<td>0.5812</td>
<td>-0.0545</td>
<td>0.0817</td>
</tr>
<tr>
<td>avg</td>
<td>-0.1796</td>
<td>0.1430</td>
<td>-0.0727</td>
<td>0.0294</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>NP</th>
<th>MIN</th>
<th>MAX</th>
<th>MEAN</th>
<th>STD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-0.2865</td>
<td>0.5232</td>
<td>-0.0557</td>
<td>0.0535</td>
</tr>
<tr>
<td>2</td>
<td>-0.6594</td>
<td>0.3816</td>
<td>-0.0709</td>
<td>0.0395</td>
</tr>
<tr>
<td>3</td>
<td>-0.2029</td>
<td>0.5494</td>
<td>-0.0588</td>
<td>0.0849</td>
</tr>
<tr>
<td>4</td>
<td>-0.2521</td>
<td>0.4999</td>
<td>-0.0613</td>
<td>0.0618</td>
</tr>
<tr>
<td>5</td>
<td>-0.1084</td>
<td>-0.0564</td>
<td>-0.0842</td>
<td>0.0071</td>
</tr>
<tr>
<td>6</td>
<td>-0.1584</td>
<td>0.0485</td>
<td>-0.0859</td>
<td>0.0147</td>
</tr>
<tr>
<td>7</td>
<td>-0.0975</td>
<td>0.1601</td>
<td>-0.0715</td>
<td>0.0229</td>
</tr>
<tr>
<td>8</td>
<td>-0.1900</td>
<td>-0.0071</td>
<td>-0.0861</td>
<td>0.0109</td>
</tr>
<tr>
<td>9</td>
<td>-0.2114</td>
<td>0.2669</td>
<td>-0.0797</td>
<td>0.0374</td>
</tr>
<tr>
<td>10</td>
<td>-0.2173</td>
<td>0.3740</td>
<td>-0.0681</td>
<td>0.0405</td>
</tr>
<tr>
<td>11</td>
<td>-0.1355</td>
<td>0.1262</td>
<td>-0.0692</td>
<td>0.0167</td>
</tr>
<tr>
<td>12</td>
<td>-0.3339</td>
<td>0.0806</td>
<td>-0.0744</td>
<td>0.0183</td>
</tr>
<tr>
<td>13</td>
<td>-0.4212</td>
<td>0.1531</td>
<td>-0.0843</td>
<td>0.0422</td>
</tr>
<tr>
<td>14</td>
<td>-0.3322</td>
<td>0.1456</td>
<td>-0.0803</td>
<td>0.0362</td>
</tr>
<tr>
<td>15</td>
<td>-0.5408</td>
<td>0.2283</td>
<td>-0.1026</td>
<td>0.0921</td>
</tr>
<tr>
<td>avg</td>
<td>-0.2765</td>
<td>0.2276</td>
<td>-0.0755</td>
<td>0.0386</td>
</tr>
</tbody>
</table>

The mean settlement over the area of a single block in the improved panel was 72.7 mm, with an average of 72.6 mm recorded in the overall area covered by the kentledge. In the natural panel, 75.5 mm was the average recorded settlement for an individual block and 76.5 mm in the surcharged kentledge region. These average settlements are very similar to each other, as was the case with the maximum measured ground settlement. However, the settlements are 5 to 10% lower than the 82 mm of maximum ground settlement recorded by the survey stakes. With a difference of only 3 to 5% more settlement observed in the surcharged area of the natural panel than in the surcharged area of the improved panel, it is difficult to attribute the variation to any single cause, as many factors contribute to the soil’s capability to limit or negate post-liquefaction induced settlement.
While the standard deviation of each block may not be meaningful because it only indicates local rotation of the individual block, taking the standard deviation of the mean of all blocks can indicate the range of settlements over the loaded area. Additionally, discrepancies in the lidar data due to the rotation of the raised cross-like surface of the concrete blocks could artificially increase the variance. The standard deviation of the mean settlement of the individual blocks in the natural panel is 12.0 mm and 10.9 mm in the improved panel.

In addition to knowing the average settlement that occurred in each area, understanding how the site settled differentially is also critical. To obtain the average differential settlement for each panel, numerous polylines were drawn in varying orientations atop the blocks in ArcGIS Pro. Discrete data points from the lidar dataset were then masked to the lines at 1 cm intervals, essentially creating a profile view across the kentledge area. Best-fit lines were then calculated for each ‘elevation view’ to represent the plane of settlement in that specific orientation. The plane of maximum differential settlement for each panel was identified through a combination of manually permutating the orientation of the lines and algorithmically calculating the differences.

Once the orientation of the plane of maximum differential settlement was identified, the average differential settlement of each masked line in that orientation was calculated following the procedure proposed by Wentz et al. (2015). They suggested that the 15\textsuperscript{th} percentile of the average settlement be subtracted from the 85\textsuperscript{th} percentile of the average settlement to obtain the differential settlement. Spacing of the lines across the kentledge varied but was in the general range of 0.2 m. With the best-fit lines produced for each ‘elevation view’, the average differential settlement across each panel was obtained and normalized for a 4 m length. For the IP, the average differential settlement was 13 mm/4 m, with the NP having an average differential settlement of 26 mm/4 m.
To corroborate the magnitude of differential settlement calculated using this method, a simplistic approach was also employed. The settlements of each block were arranged in sequential order from lowest to highest for each panel. Then, using a simple plot as shown in Figure 7.18, the difference between the $85^{th}$ and $15^{th}$ percentiles was calculated. With this method the improved panel had 11 mm/4 m of differential settlement and the natural panel had 25 mm/4 m of differential settlement. Despite the simplicity, calculating the differential settlement this way allows for major outliers to not drastically influence the results. These areas of excessive settlement are not realistic, but rather caused by the inability of the survey to accurately map the edges and raised surfaces of the concrete blocks within each panel, causing false values of settlement and heave.

![Figure 7.18: Average settlement of blocks in the IP and NP for corroborating differential settlement calculations.](image)

Another parameter, the average differential settlement with tilt removed, was calculated in a similar fashion. The tilt-removed differential settlement allows for comparison of the differential settlement across the panel with respect to a best-fit plane over the area. This was obtained by calculating the residual between the best-fit lines used for finding the differential settlement and
measured settlements from the lidar surveys. In the same fashion, the 15th percentile of the tilt-removed settlement was subtracted from the 85th percentile of tilt-removed settlement. In the IP, 12 mm/4 m of differential settlement with tilt-removed occurred and 21 mm/4 m of tilt-removed differential settlement in the NP.

Plots showing the differential settlement versus total settlement, the tilt-removed differential settlement versus total settlement, and the tilt-removed differential settlement versus differential settlement for both the improved and natural panels are plotted in Figure 7.19, Figure 7.20, and Figure 7.21, respectively. In addition, data from the large-scale testing of shallow ground improvement methods reported by Wentz et al. (2015) are also shown in these figures. While the tests were carried out at different sites, the soil profiles are similar (Traylen et al. 2018) and the blasting procedures were the same, providing a good comparison for the results of this study.

![Figure 7.19: Differential settlement vs total settlement (Wentz et al. 2015).](image)

Figure 7.20 and Figure 7.21 show that the IP has a higher proportion of stiff soil crust, while the NP has a low to moderate proportion of stiff soil crust, comparable the results discussed in section 5.2.2 for the IP prior to and following ground improvement. Because of the stiffness that
the resin injection ground improvement adds and the thicker (6 m) zone of improvement when compared to the shallow (4 m) ground improvement methods studied by Wentz et al. (2015), lower values of total and differential settlement were measured. In each plot, data from the NP are plotted as reference to show the improvement in the IP relative to the NP.

Figure 7.20: Tilt-removed differential settlement vs total settlement (Wentz et al. 2015).

Figure 7.21: Tilt-removed differential settlement vs differential settlement (Wentz et al. 2015).
7.3.4 Liquefaction-induced settlement vs. depth

Measured settlement versus depth profiles from the Sondex profilometers within the depth of resin treatment (0 to 6 m depth) are plotted in Figure 7.22 at the center of the IP and the NP for comparison. Settlement has been zeroed at a depth of 6 m to facilitate comparisons. The maximum ground settlement in the IP is about 24 mm, while it is about 46 mm in the NP. This is a reduction of about 50% when liquefaction occurred at both sites.

Using the Zhang et al. (2002) CPT-based methodology, post-liquefaction volumetric strains were calculated as part of this study and used in Equation 2-2 to predict the liquefaction-induced ground settlement versus depth at the NP and IP. With the average CPT profile soundings obtained prior to the polyurethane resin treatment in 2016 and a sounding conducted in November of 2019, the average predicted 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, which are also plotted in Figure 7.22.

The measured settlement versus depth curves are generally within the range of predicted settlements based on the Zhang et al. (2002) approach. In the IP, the predicted ground surface settlements ranged from 20 to 32 mm for the top 6 m of soil. For the NP, the range of ground surface settlements was between 48 and 63 mm, just over twice as much as expected in the IP when liquefaction occurs at both sites. Based on profilometer readings, 24 mm of settlement occurred in the top 6 m of the IP and 47 mm in the NP. For the majority of the recorded profilometer readings, the datapoints fall within one standard deviation of the mean of the predicted settlements.
Figure 7.22: Comparison of profilometer settlement vs. depth profiles with predicted settlement using the Zhang et al. (2002) CPT-based method.

Additional analysis was performed to understand how the expected free-field settlement varies with peak ground acceleration (PGA) based on a Mw7.5 earthquake event. Figure 7.23 shows expected settlements based on the 2016 average pre- and post-injection CPT data obtained, along with data gathered during the month preceding the blast tests. It is evident that minimal relaxation of the soil profile occurred in the three-year period between the resin injection treatment and the blast testing. Though the natural panel was located within 30 m or so of where the pre-improved CPT tests were performed, it appears that the selected area for the natural panel contained soils that were somewhat stiffer than the improved panel prior to treatment.

Between the 2016 data sets, a percent reduction of liquefaction settlement was calculated to provide an estimate of the efficacy of the treatment process for varying PGAs. While a higher
percentage in reduced settlement is a good indication that the treatment process functioned as intended, the predicted settlements are limited by the ultimate depth of the CPT readings, or 8 m. As noted by Traylen et al. (2017), Figure 7.23 only relates to the soils to a depth 2 m below the treated zone. If a deeper soil profile were considered, the percentage reduction would be less than what is presented which may lead to a more accurate model of anticipated settlement.

Figure 7.23: Computed free-field liquefaction settlement with varying peak ground accelerations.

7.3.5 Settlement overview

One discrepancy between the various settlement measurements is the fact that ground surface settlements between 70 to 80 mm were observed in both the NP and IP based on the auto-level surveys and ground-based lidar measurement, though only 24 to 46 mm of settlement occurred within the upper 6 m of the soil profile based on the Sondex profilometers. By deduction, this means that the remainder of the settlement manifested at the surface must have occurred below 6 m. While CPT data from before and after the installation of the ground improvement process
continue to 8 m, the data are insufficient to provide a clearer understanding of what occurred at greater depths. Continuing the predicted settlements to a depth of 8 m does not yield sufficient settlement to arrive at the ground surface settlement that was observed. Additionally, based on data obtained from the PPTs, the lower boundary of the liquefied zone was not able to be determined. Liquefaction below 10 m may have occurred and influenced the measured settlements, but it is unknown whether this was the case.

Knowing that the settlement in the treated zone between 1 to 6 m of the IP was significantly less than settlement in the NP in those same depths, yet both panels experienced similar ground surface settlements, the settlement of the NP below 6 m must be less than that of the IP.

Figure 4.4 indicates that between 7 to 9 m, the stiffness of the soil in the NP is significantly higher than that of the IP, by as much as threefold. Though settlement is not directly correlated to these parameters they are reliable indicators that the soils in this layer of the NP are less likely to settle. With the lower deck of explosives being in the stiffer layer of the NP and the softer layer of the IP, it is plausible that more settlement occurred in the IP than in the NP between these depths. Ultimately, the data are insufficient to be able to reliably compare the predicted settlement below the elevation of the treated layer to the actual settlement recorded throughout the soil profiles.

Even though the total settlement of the IP and NP were quite similar, the IP performed much better with respect to differential settlement, both with and without tilt. The IP experienced 51% of the differential settlement with tilt that the NP experienced and 58% of the differential settlement with tilt removed compared to the NP. Reducing the amount of differential settlement across the test site is a key indicator that the resin injection treatment improved the soil as intended.
7.4 Accelerometer results

Four triaxial digital accelerometers were placed by INGV on a 1-m square around the center of the NP. Additionally, four triaxial analog accelerometers were placed by BYU on a 1-m square equidistant from the NP and IP along the centerline of the test site. Each accelerometer recorded motions in the X, Y, and Z directions with time at a rate of 10 kHz. Figure 6.9 shows the relative placement of the accelerometers to the X-Y coordinate plane, with the positive Z axis being vertical coming out of the ground. Both sets of accelerometers were installed at a depth of 4 m.

The ground motions resulting from the detonations are characterized by an impulsive signal of high amplitude with a duration of about 2.5 milliseconds. The signal pulses from the deeper (8.5 m) and heavier (2.4 kg) charges produced very energetic signals that are easily recognized in the time histories. In comparison, the signal pulses from the shallower (4 m) and lighter (1.2 kg) charges have lower amplitude and are more difficult to detect in the time series.

An example of the accelerometer time series recorded by BYU in the X, Y, and Z directions is provided in Figure 7.24. This record is consistent with records obtained from the other accelerometers, where the transient spikes due to the detonations are easily identifiable for many of the blasts. The maximum peak acceleration recorded by the accelerometers equidistant between the two panels was about 142g and 249g during blast #1 (IP) and blast #2 (NP), respectively. For both blasts, the largest recorded acceleration occurred in the Y direction. Raw acceleration time history records were obtained from INGV. A typical set of acceleration time history records in the X, Y, and Z directions for a single detonation is shown in Figure 7.25. All eight accelerometers remained in good working condition throughout the duration of both blasting sequences. Accelerations were measured to $10^{-5}$ g, though some noise was present in the data. Using an IIR
high-pass Butterworth filter at 5 Hz with 4 poles and 2 passes, velocity and displacement time series were obtained by INGV researchers for each blast.

Figure 7.24: An example accelerogram from AA1 during blast #2 recorded in the time domain in the X, Y, and Z directions.

With the data obtained from the accelerometers and knowing the geometries of their locations relative to the blast sources (i.e., the two explosive charges in each blast hole), it is possible to estimate the shear strains occurring at the center of each panel during the blast sequence. Estimates of blast-induced shear stresses may also be obtained using these data by way of constitutive relationships. Following these estimations, the cyclic stress and cyclic resistance ratios may be calculated. Though this information may be useful in evaluating the similarities in
energy produced by blast-induced ground motions and ground motions from real earthquakes, it is not within the research objectives of this study at present.

Figure 7.25: Time history of accelerations in the X, Y, and Z directions from Accelerometer 1 during the 5th 2.4 kg detonation of blast #2.
8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

Two full-scale blast tests were conducted in Christchurch, New Zealand to determine the viability of polyurethane resin injection as a form of shallow ground improvement in silty sands. Blast tests were carried out in an improved panel that had been treated with resin injection and an untreated, natural panel. The test site for both blasts was in a region that experienced significant liquefaction-induced damages during the 2010 – 2011 Canterbury Earthquake Sequence and was later classified by the New Zealand government as a red zone. Liquefaction was successfully induced in both test panels, as recorded by piezometers installed in the blast circles, with subsequent ground surface settlements of 70 to 80 mm. Settlement within the top 6 m of the improved panel was half that of the natural panel.

Based on analysis of the in situ results and various recorded settlement data, the following conclusions can be drawn:

1. The lasting increase in $q_c$, $q_{c1Ncs}$, $V_s$, $MDMT$, and $K_0$ measured three years after the resin injection ground improvement process indicates that the treatment’s efficacy does not significantly decrease over time.

2. Liquefaction-induced settlement in the treated layer of the IP was 47% of the settlement recorded over the same depths of the NP. Differential settlements, with and without tilt, across the IP were 51% and 58% of the differential settlements across the NP, respectively. The
reduction in total, differential, and tilt-removed differential settlements can be attributed to the IP having a stiffer and thicker crust.

3. Total, differential, and tilt-removed differential settlements plot near the bottom of the range of data observed in the large-scale blast-induced liquefaction study sponsored by the New Zealand Earthquake Commission (Wentz et al. 2015) which involved a variety of ground improvement methods. This performance indicates the viability of polyurethane resin injection as a method for shallow ground improvement. Although the total settlement of the IP was relatively small compared to ground improvement methods from the EQC study, site variability cannot be completely accounted for. Additionally, the IP was treated down to 6 m, while the panels from the EQC study only went down to 4 m.

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 predicted settlements for the depths of interest in both the IP and NP. In the treated zone of the IP, predicted ground surface settlements ranged from 20 to 32 mm. For the same depths in the NP, the range of ground surface settlements was between 48 and 63 mm, just over twice as much as expected in the IP. Based on profilometer readings, 24 mm of settlement occurred in the top 6 m of the IP and 47 mm in the NP.

5. Partial desaturation of the treated zone in the IP, indicated by a compression wave velocity below 1,500 m/s, decreased the rate of pore pressure generation when compared to the NP. With a decrease in compression wave velocity, the factor of safety against liquefaction increases.
8.2 Recommendations for future research

No theoretical framework has been derived for predicting the efficacy and quantitative improvements that the polyurethane resin injection treatment provides. While case studies are good sources of real-life data, having the capability to use theoretical design criteria is also important. Variability in the spread of the resin during installation, spacing of the injection points, and variable soils makes it difficult to accurately anticipate the effect of the treatment. Additional testing and research may lead to more consistent results and accuracy in gauging the treatment’s effectiveness.

Another recommendation is to obtain data from in-situ test methods, specifically CPT and DMT, that extend approximately 1.5 times the depth of the lowest explosive charges prior to blast testing. Knowing the strengths of the soils below the zone of interest is important in being able to quantify any settlement that may occur outside of the targeted zone.

A final recommendation is to investigate the shear stresses and strains induced by the blasts using accelerometer time histories. Correlations between the cyclic shear strain and excess pore water pressures may be made. Additionally, estimates of the cyclic stress ratio can be calculated to predict the factor of safety against liquefaction.
REFERENCES


Lusvardi, C. M. (2020). "Blast-Induced Liquefaction and Downdrag Development on a Micropile Foundation."

Mainmark (2020). "Teretek: The most advanced way to improve ground bearing capacity and relief structures." M. G. E. N. LTD, ed.


Popik, M., Trout, M., and Brown, R. W. "Improving soil stiffness beneath pavements using polyurethane injection." *Proc., presentation at the Permeable Pavement Design and...*
Technology Session of the Annual Conference of the Transportation Canadian Association, Canada.


Sabri, M., Bugrov, A., Panov, S., and Davidenko, V. "Ground improvement using an expandable polyurethane resin." Proc., MATEC Web of Conferences, EDP Sciences, 01004.


Wentz, F., van Ballegoooy, S., Rollins, K., Ashford, S., and Olsen, M. "Large scale testing of shallow ground improvements using blast-induced liquefaction." *Proc., 6th International Conference on Earthquake Geotechnical Engineering."


