CASE STUDY REVIEW OF SEISMICALLY INDUCED LAND DAMAGE MITIGATION USING STONE COLUMN GROUND IMPROVEMENTS IN CHRISTCHURCH, NZ

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ABSTRACT

The major earthquake sequence from 2010 to 2011 caused significant damage to residential and commercial properties in Christchurch, New Zealand. Damage to major civil infrastructure, as a result of seismically induced liquefaction, lateral spreading and the associated settlements is often considered to be severe. The rebuild of infrastructure within the Christchurch area required significant ground improvement works to reduce the potential for seismically induced land damage from future earthquake events.

This paper will provide a background on the effects of seismically induced land damage and detail the use of stone column ground improvement works to reduce or mitigate liquefaction potential through ground densification. Three case studies within the Christchurch area where stone columns have been used for ground densification, with differing site conditions and installation methods will be critically reviewed and appraised, including a review of the design assumptions and lessons learned from observing construction works. The theoretical effectiveness of stone column ground improvements at these sites will then be analysed and discussed using construction compliance testing. The paper will then discuss key advantages and shortcomings of stone column ground improvements in differing soil conditions found in Christchurch and how this may apply to other parts of New Zealand.

1. INTRODUCTION

The Canterbury Earthquake Sequence (CES) from 2010 to 2011 caused severe and widespread damage to residential dwellings, commercial properties and horizontal infrastructure, such as roading and pipe networks. Seismically induced liquefaction and the associated lateral spreading and vertical settlements were considered to be severe across large parts of Christchurch, New Zealand. In order to reduce the potential for seismically induced liquefaction in future events, significant ground improvement works were required during the infrastructure rebuild in Christchurch.

Ground densification through the use of stone columns has been completed on multiple occasions throughout Christchurch. The intention of ground improvement often varied from the reduction of seismically induced liquefaction potential under buildings to the strengthening of sloping ground to reduce the potential for liquefaction induced lateral spreading. This paper will consider three case studies in Christchurch where stone columns have been installed. The three sites contain differing ground conditions, installation methods and intent of ground improvement. A review of the design of stone columns at the three sites and lessons learned from observing construction works will be completed, and the theoretical effectiveness of stone column ground improvements will be analysed using construction compliance testing. This paper will also outline the key advantages and disadvantages of stone column ground improvements, considering the differing soils conditions and installation methods, and suggest how these can be applied to the future use of stone column ground improvements, both in Christchurch and elsewhere.

2. SEISMICALLY INDUCED LIQUEFACTION

Loose sand and low plasticity silts are typically susceptible to seismically induced liquefaction. Earthquake shaking resulting in cyclic loading of soils can cause these soils to contract. This results in a transfer of normal stress from the soil matrix to porewater. Provided the soil is saturated and drainage is not possible during shaking porewater pressures will increase. The result of this transfer of stress is a reduction in the effective confining stress, and a subsequent loss of strength and stiffness within the soil.

Idriss and Boulanger (2008) provided procedure for evaluating the potential for soil liquefaction during earthquakes, and refined this method in Boulanger and Idriss (2014). The likelihood of liquefaction is determined by a combination of the in-situ soils ability to resist liquefaction and the cyclic stress being applied to the soil as a result of seismic shaking. These are commonly referred to as the cyclic resistance ratio (CRR) and the cyclic stress ratio (CSR) respectively, and the factor of safety against liquefaction is expressed as a ratio of the two:

$$FS_{liq} = \frac{CRR}{CSR} \tag{1}$$

The cyclic resistance ratio of a soil is directly related to the density of the soil. Cyclic resistance increases as density does. The cyclic stress ratio is determined predominantly by the ground acceleration and magnitude of a given seismic event and related to the vertical effective stress of the soil. Hence, denser soils or soils at greater depth are less prone to seismically induced liquefaction.

Seismically induced liquefaction can result in numerous damaging effects to buildings and infrastructure, including; the surface ejection of material due to the build-up of excess pore water pressures during seismic shaking, ground surface settlements, particularly differential settlements, due to reconsolidation of liquefied soil deposits after a seismic event and lateral spreading and the displacement of faces of a free slope due to loss of support.

3. STONE COLUMN GROUND IMPROVEMENT

Stone column ground improvement involves the installation of regularly spaced vertical columns to reduce the liquefaction potential of susceptible subsurface soils surrounding the columns. Stone columns are typically installed by inserting a probe or mandrel into the ground and laterally displacing the soil surrounding the mandrel, before removing the mandrel from the ground and replacing the void created with compacted coarse angular granular material. The amount of granular material per given area of land that is placed into the ground is depends on-site conditions and the performance requirements of the ground improvement, and is commonly referred to as the area replacement ratio (ARR).

Stone columns reduce the liquefaction potential of subsurface soils predominantly through densification of the surrounding in-situ soils, which is typically easy to verify through the use of traditional geotechnical investigations, such as SPT or cone penetration testing. Stone columns also provide other secondary improvement effects, such as stiffening of the surrounding soils and vertical drainage paths for excess pore pressure under seismic shaking. These secondary effects are difficult to verify, in the instance of stiffening, or cannot be relied up on for resilience, in the instance of drainage effects, and as such are typically ignored when designing stone column ground improvements. The effects of silty sand material clogging installed stone columns during an earthquake, and as such their lack of resilience in this event, is discussed in Alexander, Arefi and Martin (2017).

Stone columns in Christchurch, NZ, are installed using three predominant methods;

- Vibroprobe
- Full Screw Displacement
- Rammed Aggregate Piers

Vibroprobe stone columns are installed by pushing a vibrating probe from the ground surface to the target depth, outwardly displacing the in-situ soils in the process. Granular material is then bottom fed as the probe is withdrawn and compacted by re-applying vertical pressure from the probe, which further displaces the in-situ soils. This process is completed as the probe is withdrawn to the surface. Vibroprobe stone columns are known to cause significant vibration and noise and as such are better suited to ground improvement installation in open areas well away from any sensitive structures or residential development.

Full screw displacement stone columns are a relatively new stone column installation technique within the New Zealand industry. The installation process involves an outer mandrel which is screwed downwards from the ground surface, the toe of which is blocked by a reverse flight auger. After reaching the termination depth, the outer mandrel is retracted, and the auger screws out of the ground, compacting the granular material which falls through to the base of the void in the process. The primary benefits of this method over Vibroprobe or Rammed Aggregate Piers are the significant reduction in vibration and noise during the installation of stone columns, which make it highly effective in densely populated areas.

Rammed Aggregate Piers (RAPs) are a proprietary system developed by Geopier Foundation Company. The installation of the columns involves the ramming of a top driven mandrel from the ground surface to the termination depth. The mandrel is then withdrawn by about 4 feet (1200mm) to allow the release of granular material through the base, before the mandrel is re-driven downwards by about 3 feet (900mm) to compact the released granular material. This systemic up-down withdrawal of the mandrel with release of granular material from the bottom is completed to the ground surface. Outward displacement and densification of the in-situ soils are completed as a result of the compaction of the installed granular material.

4. CASE STUDIES

4.1 SITE CLASSIFICATION

4.1.1 Site A

Site A comprised the development of a multi-story residential apartment building. Two Piezocone Penetrometer Tests (CPT) were undertaken below the building footprint, in addition to a Geotechnical Borehole followed by the installation of a standpipe piezometer for the purpose of groundwater level monitoring. Additionally, a review of the available information on the New Zealand Geotechnical Database (NZGD) was undertaken, which comprised of six geotechnical boreholes. Based upon the site investigations and liquefaction assessment the site was underlain by gravelly fill material to approximately 1m below ground level, underlain by loose to medium dense liquefiable sand and silty sand to a variable depth of three to six metres. Below this liquefiable sand and silty sand the site was underlain by dense gravels, medium dense to dense sands and stiff silts to a depth of 30m+. The GNS Science Groundwater Model (2013) and measurements from the installed standpipe piezometer indicate that groundwater is located approximately 2.5m below ground level.

The liquefaction assessment completed using the two CPT investigations and the method of Boulanger and Idriss (2014) indicated that minor to moderate settlements could be expected during a major seismic event with calculated settlements in the order of 40mm in the upper 10m during a Serviceability Limit State (SLS) design earthquake and 85mm of settlement could be expected after an Ultimate Limit State (ULS) design earthquake. The site contained no significant slopes or free edges and as such was not at risk of liquefaction induced lateral spreading.

Stone columns were designed using the methods of Baez and Martin (1994) and Salgado, Boulanger and Mitchell (1997). The intention of the stone column ground improvements was to densify the sand and silty sand materials present within the upper three to six metres, in order to achieve a Factor of Safety (FoS) of 1.2 against liquefaction initiation during an ULS design event. To do so, the following installation methods were specified:

- Vibro-probe or similar: 12% area replacement rate with a minimum stone column diameter of 650mm.
- Rammed Aggregate Piers® (RAP): 8% area replacement ratio with a minimum stone column diameter of 550mm.

Rammed Aggregate Piers were selected by competitive tender process.

4.1.2 Site B

Site B comprised the development of a shallow stormwater basin for the purposes of improving land drainage capacity. 16 CPT tests were completed in the area of the proposed basin crest, in addition to shallow test pits and a review of available information on the NZGD. Shallow piezometers were installed and monitored, indicating a groundwater level of approximately 1m below ground level. Based on site investigations results, the site was underlain by predominantly clean sand. Liquefaction assessment was completed using cone penetration data, and indicated significant liquefaction potential within the upper 3m. The clean sand material increased notably in density at approximately 3.5 to 4m below ground level. The construction of the stormwater basin will result in the creation of a lateral spreading hazard for the surrounding residential properties. A ground improvement design was completed by others, based on the client’s performance specification. The design comprised of stone columns with a varying area replacement ratio of 17% to 25%, with a diameter of 850mm. Stone columns were installed using the screw-displacement method.

The intention of the specification for ground improvement was to provide a reinforced soil block which reduced horizontal displacements associated with liquefaction induced lateral spreading to less than 15mm under an ULS event. The target strength of the block was for an internal friction angle of 35 degrees, which was deemed to equate to a cone penetration resistance (q_c) of 15MPa.

4.1.3 Site C

Site C comprised the development of an emergency services building, which the client wished to maintain functionality during Importance Level Four (IL4) events. Preliminary investigations included eight CPT, one geotechnical borehole, the installation of one shallow piezometer, and a review of the readily available information on the NZGD. Based upon the site investigations the site was predominantly underlain by sand with varying fines content to approximately 20m below ground level, with a layer of finer silty material between 2m and 3m depth.

The assessment of liquefaction was carried out using CPT data. Fines content laboratory testing was completed at the site for the calibration of a C(FC) coefficient. Laboratory testing did not clearly indicate what fines content correction could be appropriate and therefore a coefficient of 0 was assumed. The liquefaction assessment indicated major calculated settlements of 200 to 280mm in an SLS event and 250 to 320mm in an ULS event. The site was approximately 150m from the Avon River (free edge) and as such the risk of lateral spreading was deemed to be minimal.

Stone columns were designed using the method of Baez and Martin (1993). The intention of the stone columns was to suppress liquefaction through ground densification to provide a factor of safety of greater than 1 under an ULS IL4 design event (1/2500 year return interval, 0.63g peak ground acceleration). The ground improvement design specified 600mm diameter stone columns on a triangular grid, spaced in a triangular pattern at 1.6m, to a depth of 11m. This equated to an area replacement ratio of 13%. An alternative design was provided by the contractor and selected by the client to install Rammed Aggregate Piers at an area replacement ratio of 8%. This was considered acceptable as the method of installing RAPs, as discussed above, causes increased stiffening effects, and can as such be installed at lower replacement ratios.

A summary of the Soil Behaviour Index (I_c) and cone tip resistance (q_c) profiles for the three sites is given in Figure 1 and 2. The proposed design depth of stone column ground improvement is also indicated on the graph of q_c.

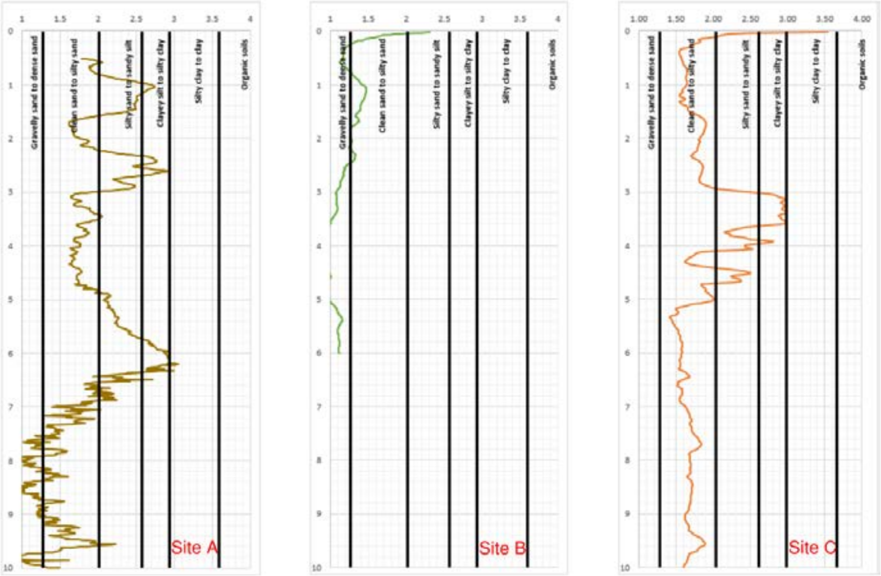

Figure 1 Soil Behaviour Index (I_c) Profiles

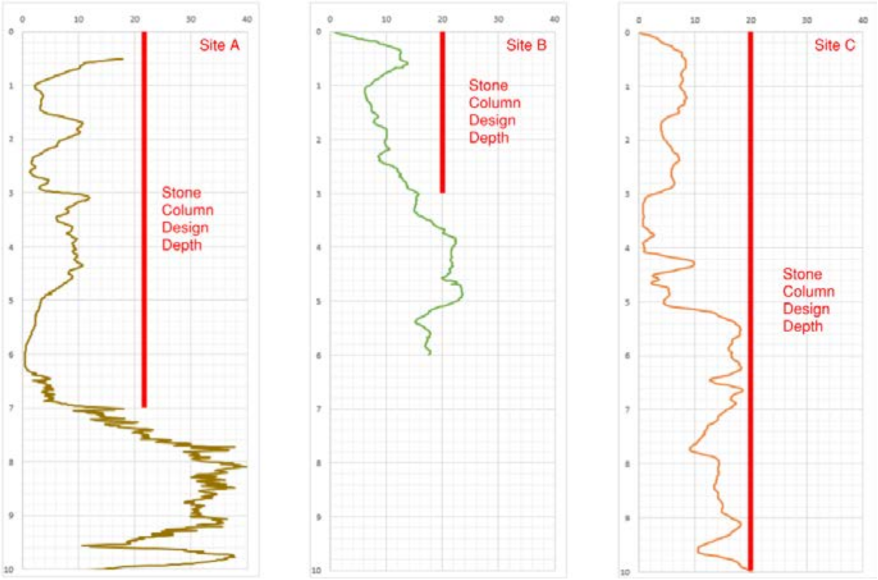

Figure 2 Cone Tip Resistance (q_c) Profiles

4.2 STONE COLUMN INSTALLATION AND EFFECTIVENESS

4.2.1 Site A

The ground improvement installed at Site A was a square grid of top driven rammed aggregate piers (RAPs). The RAPs were installed from 3 to 8m below ground level, and were keyed approximately 500mm into the natural gravel layer present. A field trial was completed, involving the installation of 25 RAPs to confirm constructability and determine whether the ground improvement approach was suitable for the site. Three cone penetration tests and cross-hole shear wave velocity testing completed following the installation of trial columns indicated that liquefaction under an SLS earthquake had been mitigated, and the liquefaction induced settlements in an ULS event had been reduced to approximately 20mm, which equated to approximately 75% reduction in settlement. In the sandier materials, the cone tip resistance post ground improvement typically increased by 3 to 4MPa. However, there was very little increase in the penetration resistance of the silty sand, sandy silt and silt materials. This level of improvement, despite not meeting the initial design intent, was considered a satisfactory level of expected future performance by the client and production RAPs were installed.

A total of 472 RAPs were completed under the proposed building platform. Throughout installation, small amounts of surface heave occurred, particularly at the edge of the building platform where the column spacing was reduced due to site boundary constraints. The drivability of the installation mandrel was used to verify the RAPs had been correctly keyed into the lower gravel layer, however the contractor had noted difficulty distinguishing the gravel from denser sand layers that were present at the southern extent of the site. In this area, it was deemed that the thickness of the liquefiable block was sufficient to meet the design intent, and that any remaining thin layers of liquefiable material were unlikely to have any significant effect.

The effectiveness of the production stone columns was confirmed using CPT. The shear wave velocity testing during the trial reflected the corresponding CPT results and production testing was continued with CPT only. Ground improvement was effective in reducing calculated SLS settlements to less than 10mm. Cleaner sand layers were typically improved to the point where liquefaction no longer occurred under a SLS or ULS event. In contrast to the success of reducing SLS post improvement free field settlement, ULS settlements of up to 40mm remained, predominantly due to the silty sand, sandy silt and silt layers not being able to be sufficiently improved. It was decided that this mitigation of SLS settlement and reduction of ULS settlement was acceptable. The structural design engineer confirmed that these settlements were tolerable for the superstructure, whilst also acknowledging that this amount of liquefaction was unlikely given previously noted limitations of assessing liquefaction in intermediary soils.

4.2.2 Site B

The ground improvement installation comprised a square grid of top-driven stone columns. The replacement ratio of the columns varied across the site, dependent on the geometric requirements of the ground improved block deemed necessary to mitigate lateral spreading. Stone columns were installed to 4m below the crest of the stormwater basin, below which dense non-liquefiable sand was present.

A field trial of column installation was deemed unnecessary by the contractor and their nominated consultant. 530 production columns were installed over a period of eight weeks. Construction observations of the stone columns noted significant surface heave at each stone column location, with a significant amount of insitu soil material ejected during the installation process, as seen in Figure 3.

Figure 3 Typical Heave from Stone Column Installation

The effectiveness of the ground improvement was confirmed using CPT testing. A total of 39 CPT tests were completed to a termination depth of 4m. The CPTs were completed in sets of three, to form a cross section of the ground improved block, and were pushed in the centre of a square of columns.

Analysis of test data indicated that surface heave caused significant loosening of the upper 1m, lowering cone tip resistances in this region by up to 5MPa. Between 1m and 3m below ground level, there was improvement in cone tip resistance between 2 to 7MPa. However the larger improvement occurred in the very loose sands. At no point in the ground improved block did the cone penetration resistance meet the specified 15MPa, across multiple areas with design area replacement ratios of 16% to 25%. The contractor attempted to rectify this by increasing the replacement ratio to 30% in the final installation area. However, quality control CPTs indicated that this area saw some even lesser degrees of improvement.

Despite not meeting the strength criterion of the specification, the ground improved block was typically non-liquefiable below the loosened upper 1m. As a result of the stone column improvements not meeting the specification and therefore not mitigating the lateral spreading hazard, further construction of gravel embankment was required.

4.2.3 Site C

The ground improvement was installed using top driven RAPs in a triangular grid. A full scale field trial of 35 RAPs were installed to confirm the suitability of the selected solution. Initially, the first three RAPs were installed to the termination depth of 11m below ground level, however subsequent RAPs were not able to penetrate past 5m to 7m. Cone penetration testing confirmed that advancing the initial RAPs had significantly densified a 2m thick layer at approximately 5m depth. This was an effect particular to the installation method using RAPs. Given the lightweight nature of the proposed super-structure and the minimum 7m non-liquefiable crust which had been confirmed through cone penetration testing in the trial area, installation of RAPS proceeded to a minimum depth of 5m, and where possible to drive to the depth of 11m, as specified in the original design.

670 RAPs were installed across the building platform. Minimal heave was noted during construction observations, and RAPs were predominantly installed into the dense sand layer at approximately 5m depth, with the occasional RAP (<5%) able to penetrate this layer and reach the original design depth.

The effectiveness of the RAPs to mitigate liquefaction under the building platform were confirmed using cone penetration testing. 13 CPTs were completed across the building platform to a depth of 12m. The improvement in cone tip resistance was highly variable based on the soil type. Clean sand material showed significant improvement and was typically non-liquefiable at all design level earthquakes. However, ground improvement was not consistently achieved in the upper 1m of the soil column. We suggest that this was due lack of confining pressure for RAP installation.

Similar to previous examples, the silty layer present at depths from 2m to 3m, showed little strength gain or improvement in resisting liquefaction susceptibility. Despite liquefaction analysis suggesting this layer was predominantly liquefiable, this was not considered a major risk due to the presence of the surrounding free draining sand and un-accounted for thin layer effects in the liquefaction analysis. Installation of stone column ground improvements resulted in a non-liquefiable crust which was typically in excess of 6m, allowing the construction of the emergency services building on a “normal” shallow foundation system.

4.3 CONSTRUCTION LESSONS LEARNT

Based on the site observations and results of the quality control testing detailed in Section 4.2 above, the following lessons learnt are considered relevant to future stone column projects, both in Christchurch and elsewhere:

- Silty sands, sandy silts and silts show limited densification using cone penetration resistance testing after the installation of stone columns. This is further complicated by the uncertainty in the actual liquefaction potential of these soils. If stone columns are to be used on sites with significant interbedded layers of silty material, the liquefaction potential of these should be confirmed through detailed fines content laboratory testing prior to the installation of ground improvement.
- Careful consideration should be given to the presence of interbedded medium dense and dense sand layers and the ability of the stone column rig to penetrate these and improve looser layers below, particularly where initial stone columns improve this layer and prevent subsequent columns to be driven through. Some improvement techniques also provide downward ground improvement and soil density increases despite columns ending short.
- Large area replacement ratios (i.e >20%) do not ensure that ground improvement will be effective and in case of relatively shallow installations are more likely to create significant surface heave, which in turn reduces the soil density. Careful consideration should be given to choosing area replacement ratios that allow for ground improvement without significantly heaving the near surface soils. Honest contractor feedback is helpful as are published case studies in similar soils.
- Stone column ground improvements for lateral spreading mitigation should be used with caution given the lack of shear strength of the individual columns themselves, in comparison to other ground improvement techniques such as in-situ soil-cement mix panels or vibro concrete columns.
- Confirmation of stone column ground improvements effectiveness is typically completed within two weeks of the installation. On sites where stone columns were installed, anecdotal evidence from contractors working after a significant delay indicate that the ground was very dense, which may indicate that the densifying effects of stone columns increase after greater time. Despite this, there is often little to no programme time available to confirm if improvement from stone columns increase over time.

5. CONCLUSION

The three sites outlined in this paper have been identified as having variable susceptibility to seismically induced liquefaction. Stone column ground improvements, using the installation techniques discussed, have been installed at each site. Although the design intent varied at each site, the overall ground improvement philosophy was similar on all sites. All designs focused on suppressing seismically induced liquefaction, with Site A and Site C intending to reduce liquefaction during an ULS event, while Site B required the reduction of liquefaction induced lateral spreading displacements during an ULS event.

At all three sites, the stone columns were shown to be effective in reducing the liquefaction potential in clean sandy soils. However, more silty soils, such as those at Site A and Site C showed little to no reduction in liquefaction potential. Site A and Site C highlighted potential difficulties installing stone columns through intermediate dense layers targeting densification of looser layers below. Site B showed the need to carefully select the area replacement ratio and carefully control heave of near surface soils. Overall careful Quality Control was important to adjust the construction to match the design intent.

6. ACKNOWLEDGEMENTS

The author would like to acknowledge the assistance of co-workers Dominic Mahoney, James Muirson and Robert Kamuhangire for their assistance during the design and construction of these projects, and to Dr Jan Kupec for his technical review of this paper.

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