Findings of the subsurface compacted rubble rafts (SCRR) pilot project for severe liquefaction mitigation in Christchurch

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ABSTRACT

This paper reports the findings of a pilot project employing the Subsurface Compacted Rubble Raft (SCRR) method for ground improvement to mitigate severe liquefaction risk. Conducted in Christchurch's "red zone" following the 2010–2011 earthquakes, the project aimed to test the SCRR's efficacy in reducing liquefaction susceptibility from Technical Category (TC) 3 to at least TC 2, meeting Ministry of Business, Innovation and Employment (MBIE) guidelines. SCRR bulbs were installed across a 180 m² site in three layers at depths of 3 to 8 meters, arranged on a triangular grid with 2-meter spacing. 111 bulbs were constructed using a 1:1 blend of AP40 and AP20 aggregates, with installation taking just 10–15 minutes per bulb. Post-installation testing confirmed significant improvements, including a 104% increase in cone penetration tip resistance. Liquefaction-induced settlement and lateral movement were reduced by 40% and 84%, respectively. Key achievements included the development of interlocked pyramidal SCRR structures, improved soil stiffness, and enhanced soil bearing capacity, making SCRR a robust and efficient ground-improvement method. The project highlighted opportunities to optimize bulb design and installation parameters for further performance enhancements. Additionally, the technology demonstrated environmental sustainability by using eco-friendly materials and recreating clean-fill land. This pilot project establishes SCRR as a valuable technology for addressing liquefaction challenges worldwide and promoting the sustainable redevelopment of earthquake-damaged areas. The project should not only stimulate new areas of research and development but also initiate contributions to the overall knowledge of earthquake engineering and ground remediation.

1 INTRODUCTION

Following the 2010–2011 Canterbury earthquake sequence, various ground-improvement and liquefaction-mitigation technologies have been implemented in New Zealand to address the resulting geotechnical challenges. These include stone columns (Earthquake Commission New Zealand, 2018), timber piling (Earthquake Commission New Zealand, 2015), ground grouting (Murashev *et al.*, 2017), horizontal soil mixing (Wansbone and van Ballegooy, 2015), and rammed aggregate piers (Wissmann *et al.*, 2015). These methods have primarily targeted areas with light to medium liquefaction vulnerability. The Canterbury region, encompassing the city of Christchurch, experienced the most significant damage during the earthquakes.

Previous research has investigated the effects of ground-improvement techniques on soil properties. Wotherspoon et al. (2015) studied shallow vibro-replacement stone columns (SCs) and rammed aggregate piers (RAPs) in Christchurch using direct-push cross-hole testing. Their findings showed that SC techniques stiffened native sand, while RAP installations enhanced soil shear-wave velocity and the shear modulus for soil between RAPs, within the RAP zones. However, these increases were not explicitly correlated with reduced settlement during seismic events. Mahoney and Kupec (2013) compared liquefaction mitigation measures across three Christchurch sites with varying susceptibilities to seismic liquefaction. These measures included composite geogrid-reinforced gravel and concrete raft foundations, stone columns, and continuous flight auger (CFA) piles. While improvements in soil resistance were analysed, settlement and lateral movement reductions were not detailed. Traylen et al. (2017) reported on-site trials using resin injection in Christchurch, where an 80% increase in cone penetration testing (CPT) tip resistance and a 50%–80% reduction in liquefaction-induced settlement were observed. Despite the promising results, the lack of correlation with the MBIE target values and technical category (TC) index depths limited the assessment of compliance with regulatory guidelines. These methods have not been effectively utilized in areas with severe liquefaction susceptibility, such as red-zoned land and high-risk TC3 zones in New Zealand, especially in relation to compliance with TC classifications and target values.

The Subsurface Compacted Rubble Raft (SCRR) (New Zealand Patent No. 728889) is a novel, patented ground-improvement technology developed to address severe liquefaction challenges (Du, 2018). It involves constructing thick artificial strata beneath the natural crust by installing rubble "bulbs" typically at depths greater than 3 m in liquefiable soils, creating a raft with a thickness of 4–6 m (Fig. 1a) (Du and Shahin, 2016; Du and Xu, 2023). This method mitigates severe liquefaction and allows for the construction of standard foundations on the treated ground. Figure 1a illustrates the profile of an SCRR-improved site.

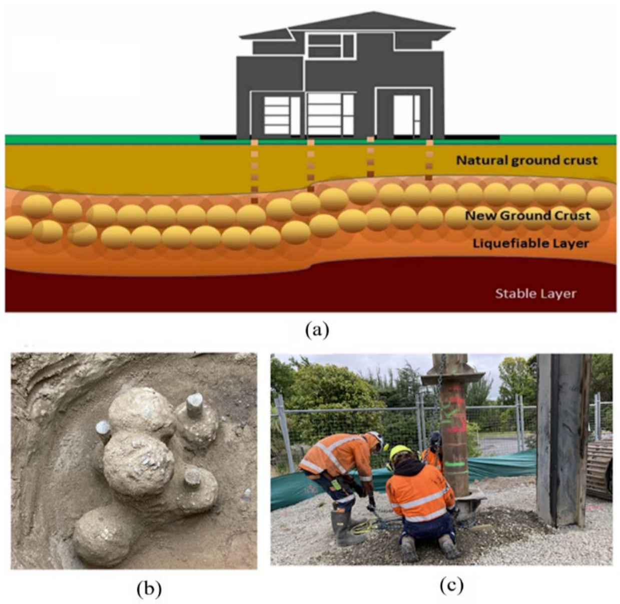

Figure 1: (a) Schematic cross-section of an SCRR-improved site, (b) Miniature SCRR bulb excavation, (c) Pilot-scale SCRR installation in progress.

A fundamental principle of ground improvement is the replacement of liquefiable soil layers with non-liquefiable materials to eliminate liquefaction risk. For extensive and thick liquefiable layers, complete replacement is impractical, necessitating a hybrid approach where the upper portion is replaced, and the lower portion is compacted with non-liquefiable material (Earthquake Commission New Zealand, 2011; MBIE, 2021b). This approach forms a thicker Ishihara crust (Ishihara 1985), creating a subsurface bridge over unstable soils to mitigate liquefaction risk. The SCRR methodology incorporates cost-effective and environmentally sustainable materials, providing a viable solution for liquefaction-prone areas.

This study evaluates the efficacy of SCRR installation for ground improvement and liquefaction mitigation through trial installations in the Wainoni red zone. Pre- and post-installation soil density and stiffness were measured using CPT, dynamic probe super heavy (DPSH) testing, dynamic cone penetration (DCP; Scala penetrometer) measurements, and multi-channel analysis of surface waves (MASW). The results were compared to quantify the improvements achieved through SCRR installation.

2 PILOT PROJECT SITE

In late 2013, EQC conducted ground-improvement trials in Christchurch's residential "red zone" suburbs of Avondale and Bexley (Traylen, 2017). These areas experienced extensive liquefaction damage during the Canterbury earthquake sequence, making them ideal testing grounds for ground improvement techniques. The success of these trials was expected to provide insights applicable to similar soil conditions elsewhere. The SCRR pilot project built upon these earlier efforts, utilizing abandoned EQC trial sites or sites with comparable subsurface characteristics to integrate prior characterisation work and enable direct comparisons of results.

Site inspections and analysis of cone penetration test (CPT) data from the New Zealand Geotechnical Database (NZGD) identified Wainoni Rd as the most suitable location (Fig. 2). The chosen site, 74 Wainoni Rd, was adjacent to two former EQC trial areas (Traylen, 2017). The selected SCRR treatment area at 74 Wainoni Rd measured 21.5 m × 8.7 m on the surface, with the subsurface treatment zone extending to 23.5 m × 10.7 m, incorporating a 1 m perimeter buffer.

The site is approximately 2.5 meters above surrounding roads and features a relatively flat SCRR installation area. Pre-installation testing included five CPT determinations, which revealed consistent subsurface conditions across the site. Additional geotechnical data from the NZGD confirmed similar conditions in adjacent areas (EQC, 2015; Traylen, 2017).

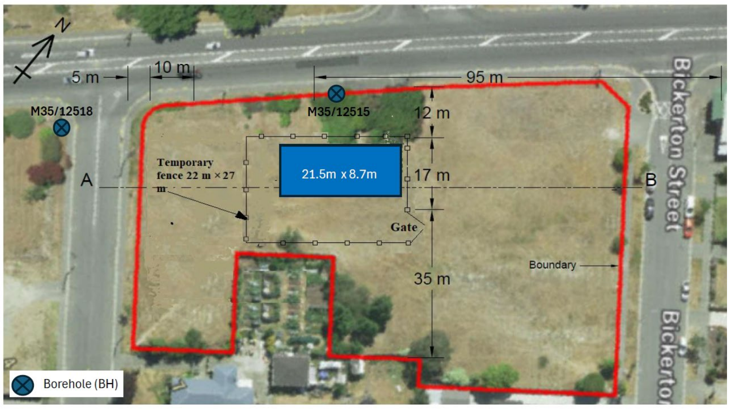

Figure 2: Plan of the SCRR pilot project site at 74 Wainoni Road, Wainoni, Christchurch. The red line encloses the site, and the blue rectangle depicts the specific treatment area for ground-improvement testing using the SCRR method (from Canterbury Maps Property Search).

The subsurface stratigraphy comprises the following layers:

- 0–3 m: Fine-grained grey sands to silty sands with some gravel, with cone tip resistance (q_c) values ranging from ~5–15 MPa.
- 3–8 m: Interbedded clayey silt and silty clay, with q_c values ranging from ~1–5 MPa.
- 8–15 m: Dense, clean sands to silty sands, with q_c values ranging from ~10–20 MPa.

Regional data from the Canterbury Regional Council Map Viewer indicated a median groundwater depth of 1.5–2.0 m near the SCRR pilot site. Variations between 1.5–3.0 m were observed at adjacent sites, such as CPT875, where ground elevation is 3 m lower. Seasonal fluctuations revealed groundwater depths of 4–5 m during the dry season. A design groundwater depth of 3 m was adopted for liquefaction and settlement analyses, consistent with historical and project-specific measurements (Traylen, 2017; EQC, 2015).

3 SCRR TREATMENT METHOD

In the SCRR method, an installation casing and a mandrel tool are inserted into the ground at regular intervals, typically following a triangular pattern, to a specified depth. Concurrently, a feed mandrel is affixed to the casing (Fig. 3, left photograph). Multi-component materials are introduced into the casing and compacted at the base of the hole. Here, they infiltrate the soil matrix during an applied vibration compaction process (Fig. 3, right photograph and Fig. 1c). The casing remains at the determined depth until the designated volume of aggregate has been introduced and SCRR bulb installation/formation is completed.

Figure 3: Rig working on a 300 mm gravel pad, showing (left) the casing and mandrel set being lowered and (right) a hopper feeding aggregate (right) at the Wainoni SCRR pilot project site.

111 SCRR bulbs were installed across the site, each consisting of 1 m³ of mixed aggregate (AP40 and AP20 in a 1:1 ratio), with a total of 111 m³ of aggregate used. The brief installation process with casing includes:

- Tool Penetration: The tool was driven to a depth of 7 m, and 0.05 m³ of aggregate was added.
- Vibration Compaction: The tool was repeatedly vibrated back to depth, compacting the aggregate until 1 m³ formed a bulb at 7 m.
- Repeat for Shallower Depths: The process was repeated for depths of 4 m and 5.5 m, with aggregate fed through a side window.

4 SCRR TEST RESULTS AND FINDINGS

A clearly defined trend of increased soil resistance (density) and stiffness is observed across all test locations of the SCRR pilot project site in a comparison of the results of pre- and post-installation in situ testing, as detailed in the following sections. The degree and extent of increase typically vary with soil characteristics, including soil depth, soil type, and pre-installation soil density.

4.1 Overall Test Results

Table 1 summarises comparisons of pre- and post-installation mean CPT tip resistance (q_c and $q_{c1N.cs}$) and surface wave velocity from MASW measurements.

Table 1: Mean percentage increases in parameter values within depth ranges of 0.5–8 m and 3–8 m.

Mean Percentage Increase in Parameter Value						
Tested Location	13 CPT q_c		13 CPT $q_{c1N.cs}$		2 MASW Lines	6 Scala
	0.5–8.0 m	3.0–8.0 m	0.5–8.0 m	3.0–8.0 m	V_s	Blows
Soil between bulbs (11)	**104%**	**117%**	**54%**	**55%**	**40%**	Increased by **27%** from 9.8 to 12.5
Gravel columns (2)	280%	788%	150%	349%		
Composite SCRR	148%	285%	78%	129%	40%	

The top 0.5 m of soil was loosened by an excavator to enable CPT and DCP penetration. Soil in the depth range of 3–8 m is the SCRR target treatment depth, with gravel bulbs installed from 4 to 7 m. CPT q_c values within the soil zone depth range (0.5–8.0 m) increased by 104% from pre- to post-installation, corresponding to an increase of approximately 15 MPa. This resulted in a mean increase in normalised equivalent clean sand tip resistance ($q_{c1N.cs}$) of 54%, representing an increase of about 46 atm. In gravel columns, CPT q_c values showed a much higher increase than those in soil, increasing by 280% and 788% in the depth ranges of 0.5–8 m and 3–8 m, respectively, with increases in $q_{c1N.cs}$ of 150% and 349%. Overall, the mean composite SCRR q_c increased by 148% and 285% in the two depth ranges, respectively, from pre- to post-installation.

The geotechnical investigations reveal that the ground in the installation/treatment area at the pilot SCRR project site was effectively improved with respect to soil resistance, density, and stiffness, and the extent of improvement (i.e., the distance over which the soil improvement occurred), with the most pronounced improvements being recorded in the CPT results.

4.2 Post-Liquefaction Settlement and TC Classification

CPT determinations (Nos. 302 to 309) were conducted 30 days after SCRR installation, allowing ground improvement to be measured (Table 2). Of eight CPT determinations, six showed improvement in soil resistance, whereas the remaining two determinations penetrated the columns formed from the filled mandrel hole to test column aggregate cone resistance. These columns were deliberately not compacted to allow them to be penetrable by CPT test equipment after installation, as our tests in previous work had shown that compacted columns were unable to be penetrated by CPT equipment.

Table 2: Computed free-field liquefaction settlement amounts (S_{v1D}) for SLS, ILS, and ULS events (worst-case scenarios) for an index depth of 10 m below ground level (MBIE, 2012). *"Pre" is pre-installation of SCRR, and "Post" is post-installation of SCRR, V-Vertical, and H-Horizontal.*

CPT No.		SLS*		ILS*		ULS*		Technical Category	Material Tested
		V	H	V	H	V	H	(TC)	
		mm	mm	mm	mm	mm	mm		
Pre	001	19	0	97	1.8	115	68	TC3/red zone	Soil
	002	10	0	61	407	84	593	TC3/red zone	Soil
	101	25	6	104	135	129	585	TC3/red zone	Soil
	102	17	4	91	155	112	703	TC3/red zone	Soil
	103	29	5	118	115	139	503	TC3/red zone	Soil
Post	302	6.2	0	50	15	97	111	TC2	Soil
	303	6.4	0	50	5	94	28	TC2	Soil
	304	6.6	0	50	52	87	352	TC2	Soil
	305	6.3	0	50	5	93	36	TC2	Soil
	306	8.7	0	60	3	93	32	TC2	Soil
	307	5	0	30	0	42	0	TC1	Column
	308	0	0	0	0	0	0	TC1	Column
	309	15	1	68	15	92	188	TC2	Soil

*SLS, 25 yr (PGA 0.13 g, M7.5), ILS, 100 yr (PGA 0.20 g, M 7.5), and ULS, 500 yr (PGA 0.35 g, M 7.5)

Table 2 reveals that of the five pre-installation CPT determinations from the site, vertical settlement amounts associated with future ULS events exceed 100 mm to a depth of 10 m, except for CPT002, which is located 10 m away from the SCRR installation area and shows a slightly lower settlement amount of 84 mm. However, CPT002 exhibits lateral movement exceeding 100 mm (598 mm), thus also categorising it as TC3/red-zoned. Moderate to severe ground damage is expected in a ULS-level event. This categorisation agrees with the landzoning type of TC3 (Christchurch) and red zone (Christchurch) obtained from the Canterbury Maps viewer (Canterbury Regional Council, 2024).

For ULS events, all the vertical settlement amounts computed from the post-installation CPT results are less than 100 mm, meeting TC2 criteria. However, the lateral movements for three CPT determinations (Nos. 302, 304, and 309) are larger than 100 mm, and these movements may be exacerbated because the site elevation is 2.6 m above the surrounding roads, with a steep slope (~45°) down to Wainoni Road. Although there is a 0.85-m-high concrete retaining wall alongside the road, the liquefaction analysis software CLiq (Geologismiki 2018) applied the assumption of level ground with free faces, not accounting for the retaining wall when evaluating the site's elevation and slope. This led to larger lateral movement calculations.

Figures 4 and 5 present plots of the computed pre- and post-installation free-field ground surface vertical settlement and horizontal movement to a depth of 10 m versus PGA, respectively. Both figures show substantial reductions in computed free-field settlement amounts across all of these design scenarios and across the entire range of considered PGAs.

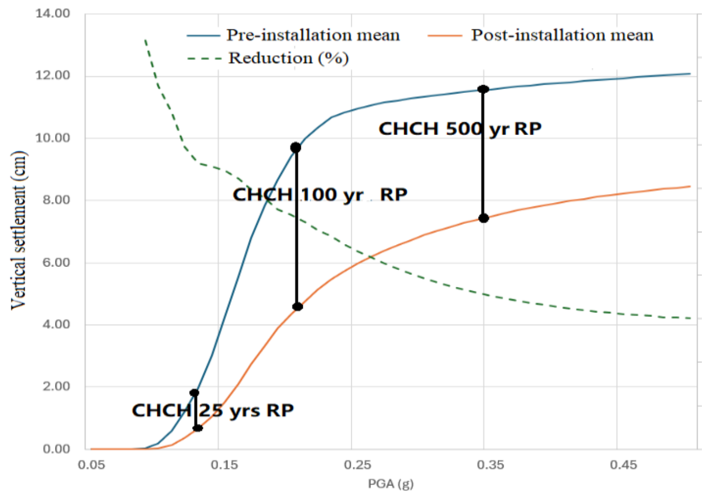

Figure 4: Amounts (pre- and post-installation) and reduction of vertical settlement after SCRR ground improvement with respect to variation in PGA. RP is the return period.

Analysis of these findings suggests that post-SCRR-installation liquefaction-induced deformations (both vertical and lateral movements) fall within the criteria of TC2, implying that the site has been upgraded from a red zone / severe TC3 to a TC2 classification and is expected to function accordingly in a future ULS event. The existence of a 2.5-m-high slope at the pilot project site may potentially induce substantial lateral displacement during an event, which could be substantially alleviated by the construction of a higher retaining wall along the Wainoni roadside.

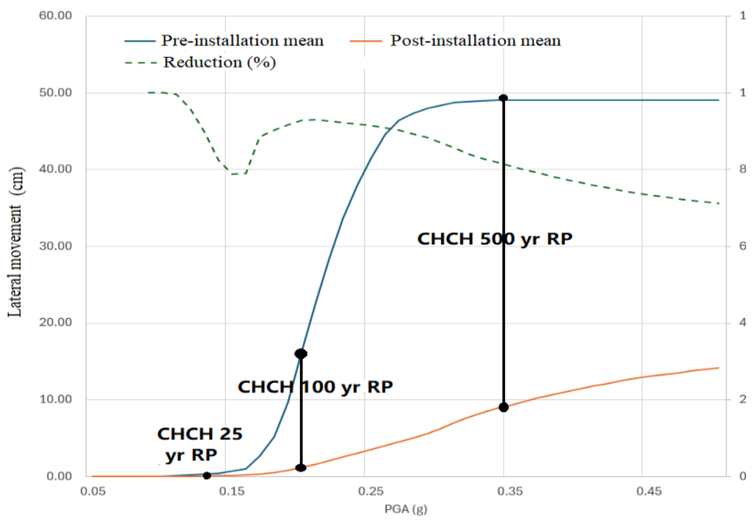

Figure 5: Amounts (pre- and post-installation) and reduction of lateral movement after SCRR ground improvement with respect to variation in peak ground acceleration (PGA). RP is the return period.

4.3 Target Value Verification

SCRR is a new technique, and there are no specific standard target values for validating the QA of its installation. However, MBIE Guidance Part C (MBIE, 2015) provides ground-improvement criteria for the deep stone column technique, which can be adopted for the initial design and verification of an SCRR to increase soil resistance above specified values. A comparison of results in Table 3 demonstrates that the soil target values from 1-8 m were successfully met.

Table 3: Comparison of CPT resistance mean values measured from 1 to 8 m depth compared with the target values q_c for clean sand, and $q_{c1N.cs}$ for all soil types excluding clay, provided in MBIE guidelines (2015a).

	Measured CPT Resistance		**MBIE Target Value**		**Measured Value Increment Over MBIE Target Value**		**Validation**
CPT	Clean sand q_c (MPa)	All soil type $q_{c1N.cs}$ (atm)	Clean sand q_c at 8 m (MPa)	All soil type $q_{c1N.cs}$ at 9 m (atm)	Clean sand q_c (MPa)	All soil type $q_{c1N.cs}$ (atm)	Overall Results
302	14.4	153.3	11.9	138	21%	11%	Achieved
303	15.8	172.4	11.9	138	33%	25%	Achieved
304	17.3	197	11.9	138	45%	43%	Achieved
305	13.6	147.1	11.9	138	14%	7%	Achieved
306	14.1	155.3	11.9	138	18%	13%	Achieved
307	19.5	228.7	11.9	138	64%	66%	Achieved
308	17.3	248.2	11.9	138	45%	80%	Achieved
309	17.8	183	11.9	138	50%	33%	Achieved

4.4 Verification of Crust Thickness

In the design of the SCRR for the pilot project site, where the natural crust was estimated to be 3 m thick (H_1) at the five pre-installation CPT locations, the SCRR was planned to be installed at depths of ≥3 m. The liquefiable layer has a cumulative thickness of 7 m (H_2) within the upper 15 m depth from the installed SCRR bottom, for the worst-case scenario of CPT103. This calculation was based on Ishihara's crust theory and graph, as detailed in the SCRR design description by Du et al. (2023). Then, the thickness of the SCRR, H_s, can be calculated as follows:

First, locate H_2 = 7 m in the curve that matches the shaking value of 0.35 g (worst case scenarios) (Fig. 6), which gives the new total crust thickness H_1' of 7.8 m. Then, the minimum SCRR thickness is computed as:

$H_s \geq (H_1' - H_1)$= 7.8 − 3 = 4.8 m, that is, $H_s \geq$ 4.8 m

Therefore, the lower interface of the installed SCRR was designed to be positioned 8 m below ground level, and the number of SCRR layers was 3. This design is conservative, considering the varying ground conditions, and the calculation is based on the worst-case scenario of CPT103.

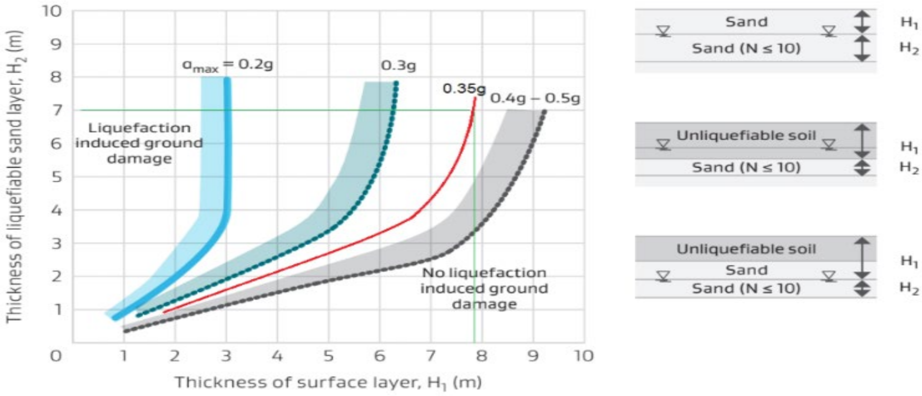

Figure 6: Estimation of SCRR crust thickness (where N is the blow count value of the SPT), adapted from Module 3 of MBIE (2021) and Ishihara (1985).

4.5 Configuration of the SCRR Interlocking Framework and Pyramidal Structure

This section examines how the bulbs are configured to form the SCRR and describes its structural mechanisms and features of the SCRR. It also discusses the bulb dimensions, and the extent of soil densification associated with SCRR installation.

Figure 7 shows three typical CPT comparison pairs for identifying the suitable depth of bulb installation by identifying ranges of substantial increases in CPT q_c from pre- to post-installation. It is observed that the vertical depth sections with substantial increases in q_c are located at depths of approximately 2, 4, 5.5, 7, and 8 m. The bulbs were installed at depths of 4, 5.5, and 7 m. These depths are analysed and interpreted in this section, whereas the changes at 2 and 8 m are interpreted in Section 4.10.

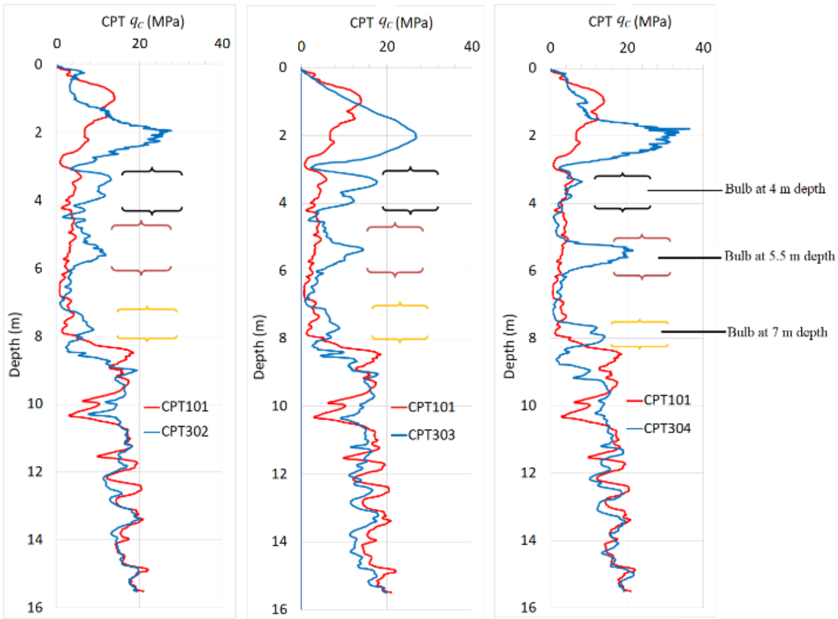

Figure 7: Comparison of post-installation CPT results with pre-installation CPT results for identification of bulb installation depths.

Figure 7 reveals that the actual depths of the bulbs designed to be installed/placed at depths of 4, 5.5, and 7 m varied within approximately ±0.5 m in the vertical dimension. The 4-m-depth bulb at CPT 304 has a low q_c value, possibly indicating less gravel feed and/or compaction at that point.

The SCRR installed at the pilot project site has a three-layer structure configured with layers of bulbs at 4, 5.5, and 7 m. Each bulb in the 5.5-m-depth layer sits on top of the mid-point of the three bulbs in the 7-m-depth layer, forming an upright pyramid, and below the three bulbs in the 4-m-depth layer, forming an inverted pyramid, except for the bulbs located along the four edges of the treatment area. Each bulb in the 4-m-depth layer sits on top of the mid-point of the three bulbs in the 5.5-m-depth layer, except for the bulbs located along the four edges. The pyramidal structure introduced is illustrated in Figure 1b. However, it should be noted that the full-size bulbs installed at the pilot project site are 10 times larger than the miniature bulbs shown in Figure 1b.

The above analysis presented the configuration of the SCRR interlocking framework and pyramidal structure. It is also necessary to describe the bulb environment and investigate the forces acting on bulbs from the surrounding soil/geology.

4.6 Post-installation Soil Densification Extent and Bulb Diameter

The diameter of the zone of densified soil was able to be calculated using Equation (1):

$$R = \sqrt{h^2 + d^2} \quad (1)$$

Where, R is the radius of the zone of densified soil surrounding the gravel core (in black) (m); d is the horizontal distance of the CPT vertical profile to the nearest bulb axis (m); h = half of the secant, that is, the thickness of densified soil (m), denoting the vertical dimension of the zone with the evident increase of q_c values compared with pre-installation measurements.

By applying Equation (1), the diameters of the installed bulbs were able to be determined (Table 4). Table 4 reveals that the SCRR bulbs attained an approximate size of a 2.4 m bulb, including an aggregate core in the centre with a diameter of 1.2 m, and a zone of densified soil surrounding the core.

Table 4: Bulb installation diameter calculated by CPT measurements for bulb installations at depths of 4, 5.5, and 7 m.

Pre-installation test	Post-installation test	Bulb at 4 m depth	Bulb at 5.5 m depth	Bulb at 7 m depth	Material tested
CPT101	CPT302	2.4 m	2.6 m	2.2 m	Soil between bulbs
	CPT303	2.4 m	2.9 m	2.2 m	Soil between bulbs
	CPT304	2.4 m	2.6 m	2.4 m	Soil between bulbs
	CPT307	3.7 m*	2.2 m	1.7 m	Column gravel
CPT102	CPT305	2.3 m	Insignificant	2.6 m	Soil between bulbs
	CPT306	2.4 m	Cone slid away from 1 m depth	Cone slid away from 1 m depth	Soil between bulbs
	CPT308	2.4 m (estimated)	Early refusal	Early refusal	Column gravel
CPT103	CPT109	2.3 m	2.1 m	Insignificant	Soil between bulbs
	Mean	2.4 m	2.5 m	2.2 m	Mean 2.4 m

*An outlier of 3.7 m was excluded from the averaging calculation.

4.7 Ground Heave and Lateral Movement

The average heave across the SCRR installation area was only 15 mm, accompanied by a mean lateral movement of 32 mm. Considering the site dimensions (10.73 m × 23.5 m), these movements are deemed relatively minor. Of note, the majority of both heave and lateral movement, approximately 80%, occurred on the slope side facing Wainoni Road, where the slope angle is approximately 45°.

4.8 Attainment of SCRR Ground Improvement Mechanisms

The pilot project results confirm that the primary mechanisms underpinning SCRR ground improvement are soil densification, lateral stress redistribution, and structural reinforcement. Auxiliary mechanisms, including soil displacement (replacement), and drainage enhancement, require further study to evaluate their specific contributions to overall ground improvement.

4.9 Layers of Maximum Soil Densification and Soil Loosening

Besides the abovementioned results showing ground improvement from SCRR installation, two unexpected findings were identified.

4.9.1 Layer of maximum soil densification at 2 m

The marked increase in q_c identified at ~2 m depth was initially attributed to mandrel compaction during SCRR installation. During SCRR installation, below depths of approximately 2 m, the soil is interpreted to have been propped and pushed upwards, trying to heave. Concurrently, the soil above 2 m was compacted and compressed by heavy machinery weighing 75 tonnes. These combined effects of SCRR installation generated a maximally densified soil layer at ~2 m (Fig. 8). The formation of this layer is a result of both upward and downward processes, in contrast to conventional dynamic compaction, which operates only downwards.

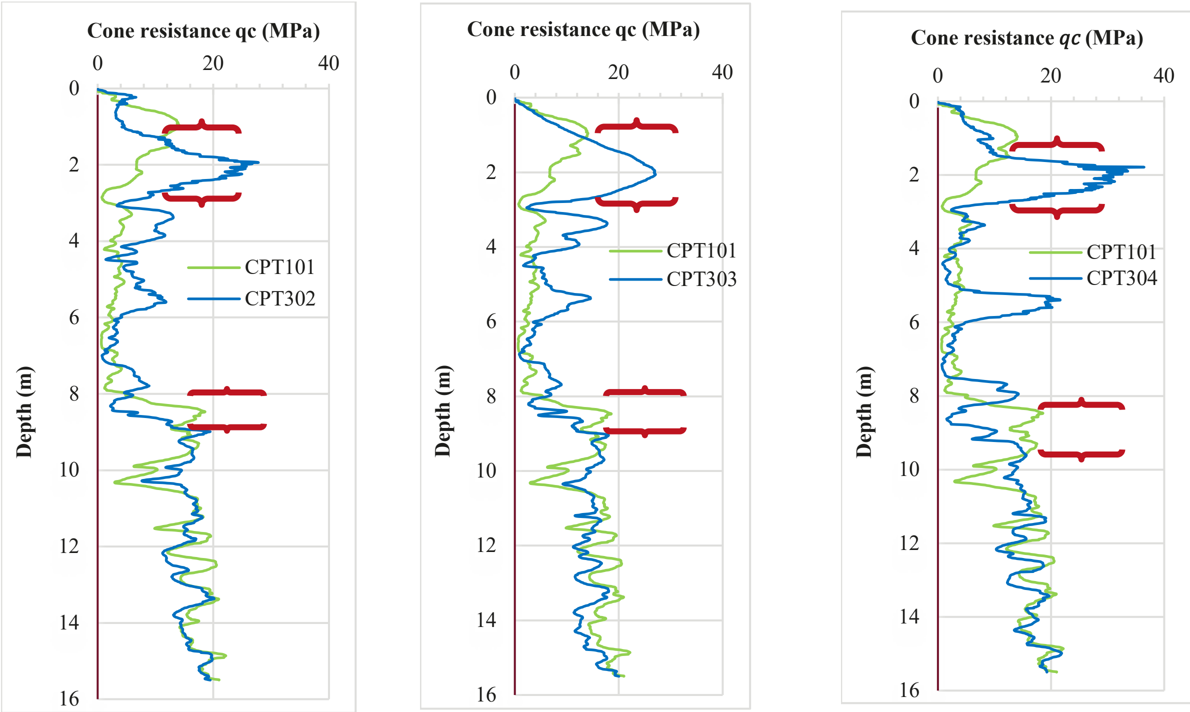

Figure 8: Three typical vertical profiles of q_c variation for CPT302–304 (post-installation measurements) and CPT101 (the corresponding pre-installation measurement).

4.9.2 Local Soil Loosening at 8.2 m Depth below SCRR zone

Out of eight CPT measurements, six identified loosened soil zones, while CPT 308 showed early refusal, and CPT 309 recorded no observations. The thickness of soil loosening zones ranged from 0 to 1.4 m. Most locations exhibited minor decreases in q_c values, except for CPTs 304 and 307. The site's heterogeneous geology suggests loosened zones occur as lenses or pockets rather than continuous layers.

To mitigate or prevent soil loosening caused by localized liquefaction,

- Extending SCRR installation to approximately 8 meters, reaching soil layers with q_c values exceeding 15 MPa, or
- Employing hydraulic static piling machines for SCRR installation, or
- For sites with extensive loose soil layers of significant thickness, implementing extended SCRR-retaining structures is one of the alternatives, however, detailed design, construction, and implementation of such SCRR retaining structures fall outside the scope of this paper.

5 DISCUSSION AND FURTHER INVESTIGATION

The pilot project represents the inaugural use of SCRR technology, detailed in this report showcasing the successful upgrade of red-zoned land to a TC2 classification. However, several remaining issues and recommendations are outlined in this section, necessitating further investigation.

SCRR technology requires ongoing refinement, including integrating international best practices and addressing limitations. Further research is needed to understand the maximum densification layer, optimize installation parameters, and explore alternative installation methods like hydraulic static piling. MBIE Target Value methods require refinement, focusing on soil between gravel columns. Gravel permeability and local liquefaction mitigation strategies, such as SCRR retaining structures, warrant further investigation. Tooling advancements are crucial for enhancing SCRR installation efficiency.

6 CONCLUSIONS

The pilot project at 74 Wainoni Road successfully validated SCRR's effectiveness in mitigating liquefaction and improving soil properties, meeting MBIE (2015b) and Ishihara Theory (Ishihara 1985) standards. SCRR installation achieved TC2 level performance, with potential for TC1 through parameter adjustments. While primarily suited for cleared sites, SCRR's robust framework, demonstrating significant lateral resistance, suggests potential over-conservatism in current validation standards compared to stone columns. Tooling advancements and refined QA/QC procedures, developed during the project, are crucial for enhancing SCRR's efficiency. The project confirms SCRR's promising capabilities in addressing severe liquefaction, with ongoing refinements expected to further expand its applicability in various geotechnical challenges.

7 ACKNOWLEDGEMENTS

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