The use of vibro-replacement on the Ngaruawahia Section of the Waikato Expressway to mitigate potential seismic liquefaction

N Wharmby & L Clark
Brian Perry Civil, Auckland, NZ.
nickw@fcc.co.nz (Corresponding author)

J Johnson
Beca Ltd, Auckland, NZ
james.johnson@beca.com

Keywords: Ground improvement, Vibro-replacement, Seismic mitigation, Liquefaction

ABSTRACT

The design and build of the 12.3km Waikato Expressway – Ngaruawahia Section was awarded to the Fletcher Construction Company by the NZTA. The project includes five local road overbridge structures and two state highway bridges; one over the Waikato River and the other an underpass to link into the future Hamilton section. The geology of the area is dominated by Piako Subgroup (Recent Alluvium and Hinuera Formation) overlying the Walton Subgroup. The Piako deposits comprise variable layers of sands, silts and clays and mixtures thereof.

For both the state highway structures the design requirement to adopt a 1 in 2500 year ULS seismic event necessitated some form of ground improvement due to the presence of layers of liquefiable loose to medium dense sands, silty sands and silts combined with a high ground water level. A number of options were considered, including predominantly structural options, but a vibro-replacement ground improvement methodology was ultimately adopted. This paper describes the:

- Ground improvement options and selection process
- Design methodology adopted for the vibro-replacement
- Construction methodology trials and evaluation
- Lessons learnt during construction
- Testing and design verification

The full scale trials performed using different installation methods provides a valuable insight with regard to the effect of the construction methodology adopted. The successful use of vibro-replacement on the project as liquefaction mitigation for the sand, silty sand and silt mixtures is an indication that this form of treatment can be more widely applied to provide a ductile ground improvement system.

1 INTRODUCTION

The $248 million Ngaruawahia Section of the Waikato Expressway is expected to be completed in late 2013. Fletcher Construction has been tasked with delivering this project, with design provided by Beca and Parsons Brinckerhoff.

The Ngaruawahia Section will connect the future Huntly Section to the north and to the Te Rapa and future Hamilton sections in the south. The project consists of approximately 12.3 km of four-lane expressway crossing potentially liquefiable soils and peat.

Five local road over bridges and two state highway bridges were designed as part of the project. Both state highway bridges were required to be designed for a 1 in 2500 year ULS seismic event. Ground displacements associated with these bridges were required to be limited to 20mm due to their sensitive nature. This paper is focussed on the Hamilton Underpass structure and describes the options considered during tender design and the journey through detailed design to construction verification.

2 GROUND IMPROVEMENT OPTIONS

With the driving force of the approach embankments during seismic events, large displacements were predicted that could not be reasonably tolerated by the bridge structure. During development of the tender design consideration was given to a number of ground improvement methodologies ranging from those that improve the strength of the soil to those that are structural elements that reduce the load applied to the soil as a function of their strength and stiffness to prevent liquefaction during the design seismic event. Table 1 provides a summary of some of those considered.

Table 1: Available Ground Improvement Options for Hamilton Underpass

Type of Ground Improvement	Overall Improvement Mechanism		Overall System Ductility	Relative Cost	Reason Not Developed
	Soil	Structure			
Dynamic Compaction	✓✓✓✓✓		✓✓✓✓✓	$	Soil + Depth
Vibrocompaction	✓✓✓✓✓		✓✓✓✓✓	$$	Soil
Dynamic Replacement	✓✓✓	✓✓	✓✓✓	$$	Depth
Vibroreplacement	✓✓✓	✓✓	✓✓✓	$$$	Selected
Deep Mass Soil Mixing		✓✓✓✓✓	✓✓	$$$$	Depth + Cost
Deep Soil Mix Columns		✓✓✓✓✓	✓	$$$	Rejected
Deep Soil Mix Walls		✓✓✓✓✓	✓✓✓	$$$$$	Cost
CFA / DFA Piles	✓✓	✓✓✓	✓✓	$$$	Cost
Timber Piles	✓✓	✓✓✓	✓✓	$$$	Cost
Containment Walls		✓✓✓✓✓	✓✓✓	$$$$$	Cost

Improving the strength of the soil requires an appropriate methodology to treat the given soil types and depth. Whilst the sands are readily treatable by dynamic compaction or vibrocompaction the silty sands and silts mixtures are unlikely to be improved. The depth of treatment of up to 12m is beyond the 5m that can be achieved with dynamic replacement and mass mixing. Based upon the cost of the remaining practical solutions, deep soil mix columns were the priced option at tender stage.

One key aspect of the Principal’s Requirements that was particularly applicable to the bridge structure seismic design was that the structures needed to be able to withstand a “significantly” greater event than ULS without failure. This would appear logical given the uncertainty surrounding seismic loading and analysis coupled with the use of displacement based geotechnical designs approaches.

The residual strength of individual unreinforced, 1MPa to 3MPa, 0.6m diameter soil mix columns is negligible once the ultimate capacity is exceeded. The capacity of a vibroreplacement solution is derived from the improvement of the surrounding ground and the strength of the 0.8m to 1.0m diameter gravel column. The capacity is only significantly reduced if there is a commensurate reduction in effective column area or soil strength following the seismic event. Trials of soil mix columns failed to achieve strength and ductility requirements at economic cement content and area replacement ratios; this lead to the adoption of a vibroreplacement solution.

3 GROUND IMPROVEMENT DESIGN

3.1 Geology

The geologic profile at Hamilton Underpass consists of Hinuera Formation overlying Walton Subgroup. The Hinuera formation comprises 11 to 14m of loose interbedded sands and silt, underlain by 2.5 to 3.5m of organic silt and clay. The underlying Walton Subgroup soils consist of interbedded loose to medium dense sands and silt, with lenses of firm to hard organic soils. The soils become gradually stronger with depth, becoming dense to very dense at 20m depth, consistent with the Karapiro Formation. The geologic profile is shown in Figure 1.

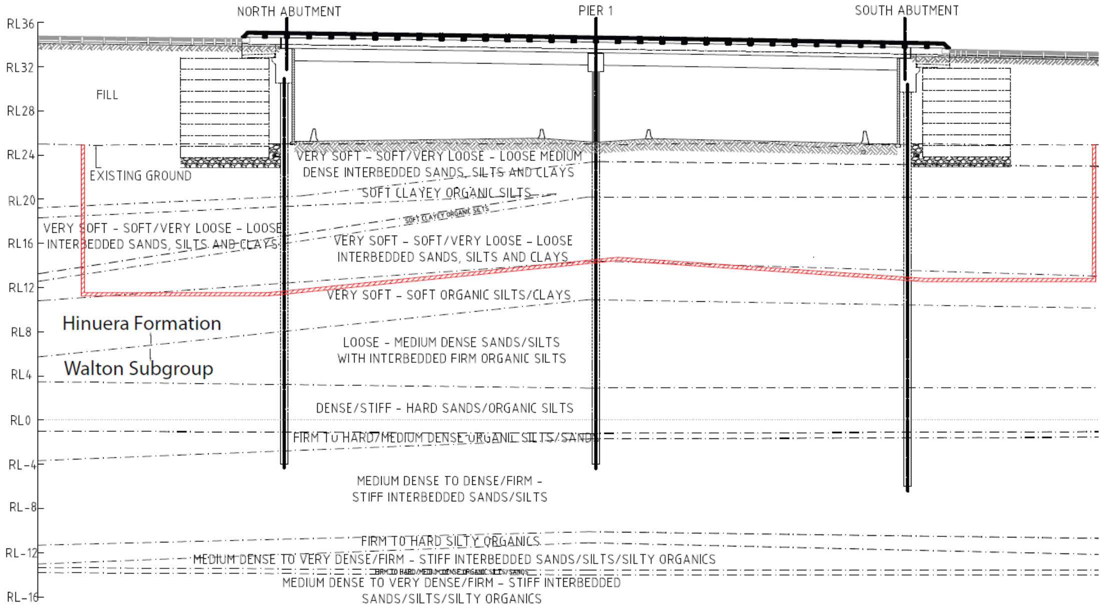

Figure 1: Geologic Profile

The ground water level is very high across the length of the project, typically being encountered between 1m and 2m depth. At the Hamilton Underpass ground water was encountered at 1m depth.

3.2 Liquefaction

Liquefaction analyses were carried out at the bridge location. The assessment was based on the NCEER proceedings, utilising both CPT and SPT data. Liquefaction was identified to be possible under the ULS seismic event between 1 and 11m depth and 15 and 18m depth.

3.3 Design Approach

The Hamilton Underpass Bridge crosses SH1 at an angle of 55 degrees on fill embankments approaching 10m in height. The severe skew of the bridge made the structure very sensitive to displacements in any direction.

The design of the vibroreplacement solution was carried out using the recommendations of Baez and Martin (1993). This method requires the shear stiffness ratio between the soil and included stiff element (Gsc/Gc) to be known. The stiff element is assumed to act only in shear which is reasonable when considering the granular column formed by the vibroreplacement. Whilst the stiffness of the ground can be determined by using well established relationships for CPTs, the stone column stiffness had to be estimated. The design strategy comprised the adoption of a Gsc/Gc ratio of 6.9 and full scale trials to refine the design and mitigate risk.

The use of vibroreplacement columns also allowed provision for densification of clean sands. This was included by allowing for a twofold improvement in the CPT tip resistance, qc, of clean sands.

The final solution allowed for a “Z” shaped area of vibroreplacement columns, with landscape fill toe weights at either side of the approach embankments. Basal geotextile was also included in the approach embankments to help limit displacements in the longitudinal direction. The solution is shown in Figure 2.

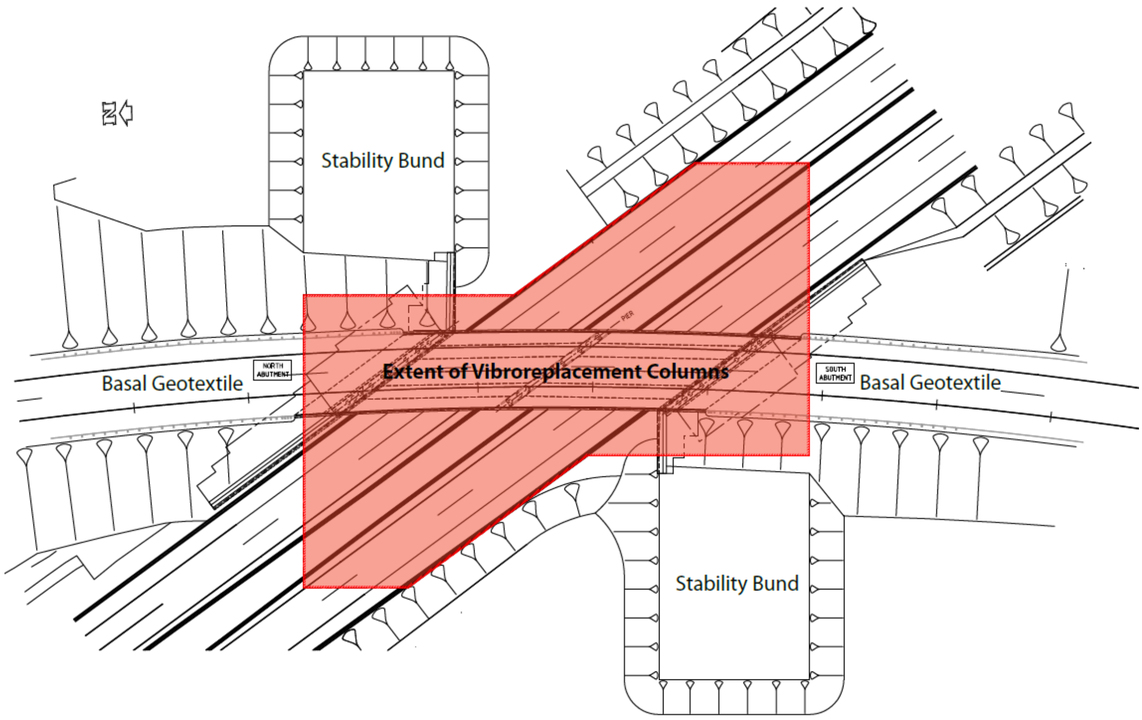

Figure 2: Plan of ground improvements at Hamilton Underpass

4 CONSTRUCTION TRIALS

Given the design requirements and the interaction of vibroreplacement process with the surrounding ground it was necessary to perform trials to confirm the level of soil densification and stone consumption given the energy applied and nature of the ground. Typically pre-treatment CPTs are used to define the soil profile and provide a benchmark with post-treatment CPTs used to measure the improvement. The vibroreplacement process results in the formation of a stone column; the use of coring down the stone column and SPTs is problematic but can provide an indication of their strength.

With significant zones of silty material (Fr > 1.0, Ic = 2.0 to 2.6) that will not be improved sufficiently by the vibroreplacement process the design relied on the strength and stiffness of the stone column. Construction methodology trials were performed using both a conventional vibro-probe and also a using a vibro-hammer and casing with a trap door.

The casings methodology uses a PTC50 piling vibro hammer with a 300kW power pack; a powerful unit is required to mobilise the casing mass and surrounding soil. This methodology has the ability to construct a column of consistent diameter.

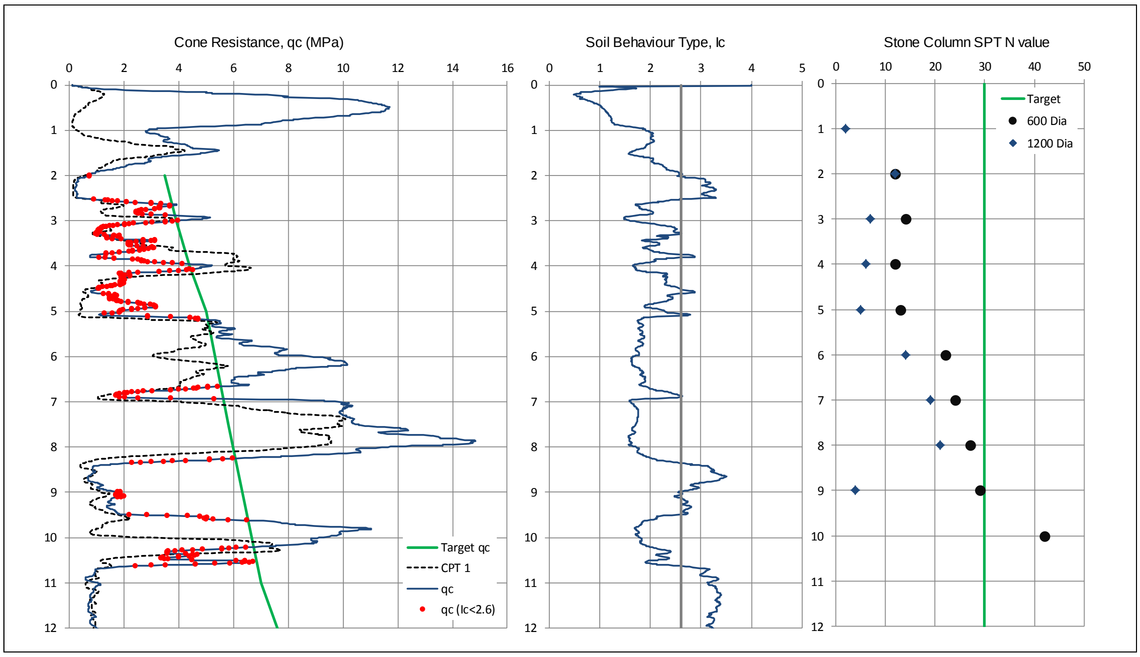

Figure 3: Test results from columns installed using casing methodology

CPT testing between the columns and coring down the stone column to facilitate SPT testing was performed. As can be seen in Figure 3 the cone resistance (qc) in some layers has increased due to the soil displacement as the casing is installed. However, the column SPT N-value is below the target average value of 30 required for both the 600mm and 1200mm diameter columns.

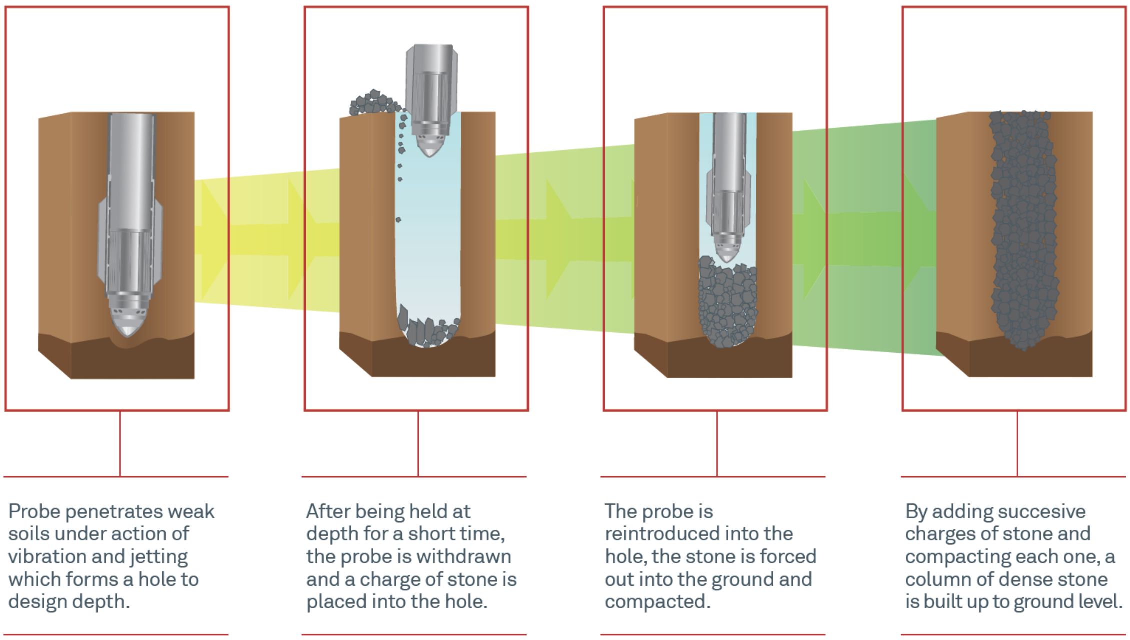

Figure 4: Vibroreplacement methodology

For the conventional vibrocompaction trials a 55kW electric probe with 7.6mm amplitude at 50Hz and centrifugal force of 146kN was used using the wet top feed methodology. Whilst the power, amplitude and centrifugal force are lower than that of the PTC50 vibro hammer the vibro probe is significantly more efficient at compacting the surrounding ground. This is because the energy is applied directly and locally to the soil and the stone introduced. The probe penetrates the stone displacing it laterally and vertically resulting in larger diameter columns in weaker soils due to the level of confinement. The basic methodology is described in Figure 4.

To ensure the design criterion could be achieved and to optimise the construction methodology trial areas comprising a 4 by 4 grid of vibroreplacement points were installed with area replacement ratios (Ar) of 17%, 20% and 25%. CPTs were carried between the vibroreplacement points and SPT N-values down the centre of the column. Analysis of the results was predominantly based upon qc, Ic, and N. A target Ar = 20% was adopted based upon this and reasonable level of engineering judgement given the interpretation of CPT data in layered soils and the effect of drilling in to the columns.

5 TESTING AND VERIFICATION

The same equipment and installation procedures were adopted for the production vibroreplacement and the testing regime as set out in Table 2 was used to verify the design assumptions.

Table 2: Testing and Verification

Testing	Purpose of testing	Frequency
SPT down the centre of vibroreplacement columns	Confirmation of design stiffness	First 5, then next 10 out of 50 followed by 1 in 35 if adequate and consistent results were obtained or 1 in 25 if not.
CPTs between columns	Confirmation of stiffness of the soil and any densification achieved	1 per 200m^2
Particle size distributions and Plasticity Index	Confirmation of permeability of vibroreplacement columns and drainage blanket	Stockpile Size: 0 to 400m3 – 2 tests 400 to 1,500m3 – 3 tests 1,500 to 4,000m3 – 4 tests >4,000m3 – 1 test for each additional 1,000m3

A specific target qc or SPT N-values for the respective soil and column stiffness is not truly relevant for this type of ground improvement. The soil type needs to be assessed when considering qc and the soil stiffness when considering the SPT N-value. Figure 5 provides an example of the evaluation process adopted. The shear modulus of the soil (Gs) is calculated from CPT data and used to generate specific target SPT N-values in the column based upon the required design shear modulus ratio (Gsc/Gs).

Overall the testing confirmed that the construction had generally met the design requirements and assumptions. There were some thin isolated lenses of potentially liquefiable material following the installation of the columns. However, the suitably robust design could accommodate this and the resulting displacements calculated.

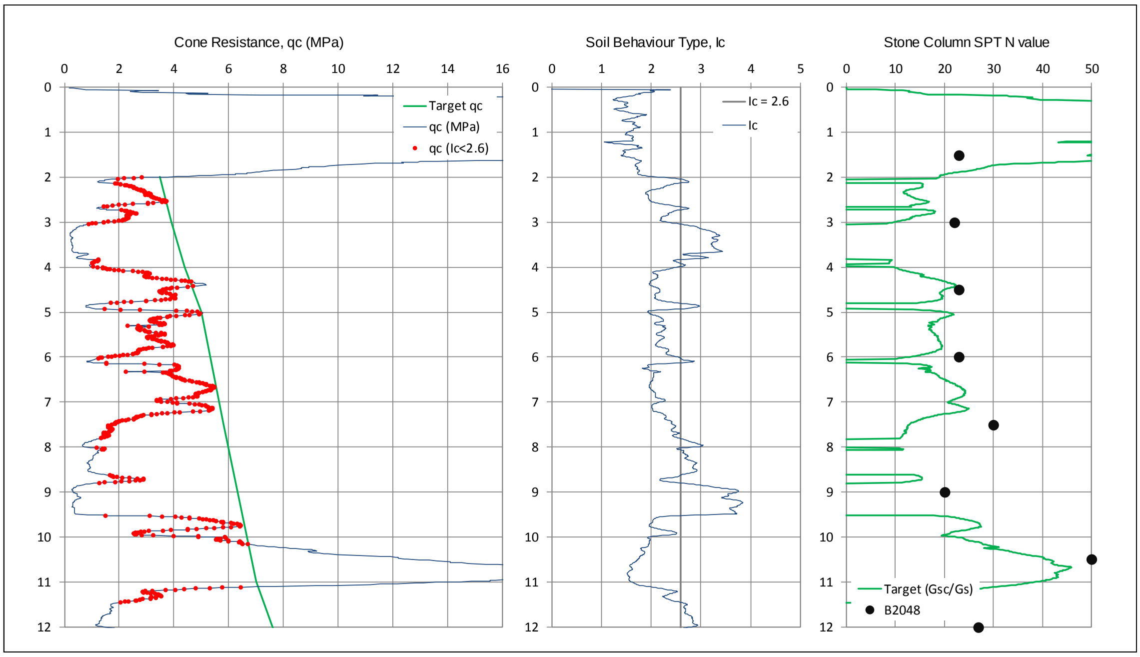

Figure 5: Vibroreplacement Verification

6 LESSONS LEARNT

There are many lessons learnt even on the design and installation of the ground improvement on the one bridge structure but the key elements are as follows:

1. The ductility and residual strength of a ground improvement solution needs to be considered particularly with ULS seismic load cases
2. The use of a piling vibro hammer and casing technique requires a high level of vibratory energy to displace and mobilise the soil; plant is required operate above normal duty cycle levels
3. The vibro probe applies the energy locally and directly to the soil and stone; a stronger / stiffer stone column formed when compared to that formed using the casing methodology.
4. Soils with an Ic > 2.0 or Fr > 0.8 cannot be reliably improved using vibratory techniques hence reliance on the column strength is required.
5. The stone column strength or stiffness achievable is limited by the confinement afforded by the surrounding ground (overburden pressure and strength).
6. Drilling and SPT testing down a column is problematic particularly when the column is of a small diameter and poorly confined.
7. Due to the interaction of vibrocompaction with the ground full scale trials are essential to verify design assumption and refine the construction methodology.
8. Detailed analysis of CPT test data combined with engineering judgement is required to evaluate the ground improvement achieved

7 CONCLUSIONS

There are a large number of options available to form a zone of improved the soil and mitigate seismic effects such as liquefaction; these range from improvement of the in-situ soils to structural elements that essentially attract the load. The most cost effective are those that improve the soils in-situ, however, soils with an Ic > 2.0 generally cannot be improved to a sufficient level, so the capacity of the "structural" elements needs to be considered.

The use of a design and construct contract allows the necessary construction experience to be incorporated into the design which is particularly advantageous when considering what can be achieved with ground improvement technologies. Even with this input, the value of full scale trials to confirm the methodology and the performance requirements can be achieved should not be underestimated. Vibroreplacement trials were performed to refine the construction methodology and finalise the target replacement ratio.

Layered sands, silty sand and silts present challenges when reviewing CPT and other test data as part of the ground improvement verification process. Detailed analysis of the data and clarity around the design philosophy are required to develop an effective review system to which engineering judgement can be applied. Assessment of shear modulus from CPT data and use of the shear modulus ratio to generate specific Target SPT N-value was successfully adopted to provide timely review for the vibrocompaction works.

Figure 5: Vibroreplacement installation

The risks pertaining to the ground improvement design and construction at the Hamilton Underpass structure were effectively managed and vibrocompaction ground improvement methodology successfully implemented.

ACKNOWLEDGEMENTS

The Authors would like to acknowledge the NZTA and their advisors for the opportunity to build the project and Keller Ground Engineering for their technical assistance.

REFERENCES

Baez, J. and Martin, G. (1993) Advances in the Design of Vibro System for the Improvement of Liquefaction Resistance, Proc. Symposium on Ground Improvement, Vancouver, Canada.