N.P. López-Acosta et al. (Eds.)

doi:10.3233/STAL190293

Vibro-Replacement for Challenging Soil Conditions in Port Structures - Recent Cases from Caribbean Area

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Abstract. The use of deep vibratory methods for the improvement of bearing capacity, settlement reduction and liquefaction mitigation of weak soils that are unsuitable as foundation for offshore structures dates back over more than 50 years. During this long period of application, a lot of experience was gained with this technology and enormous progress was achieved pushing forward boundaries and limitations for its application. The continuous development has been experienced not only regarding design methods and standards, but also equipment to carry them out in practice. In the last decade, the special features of vibro-replacement have been used in numerous offshore and port projects in Latin America. This paper deals with selected details of some of those projects, and describes the capability of vibro-replacement method to meet the typical requirements for port projects. Based on those case histories, the general framework for the design and execution of vibro-replacement aimed to carry out rehabilitation and expansion of port projects are outlined, with special emphasis on the impact of life cycle of the projects according to different seismic conditions and specific technical requirements

Keywords. Liquefaction mitigation, vibroreplacement, port, infrastructures.

1. Introduction

Countries in Caribbean Area have seen increasing growth in infrastructure development in the last decade. The presence of large deposits of weak soils of varying types and considering the °design situation under seismic conditions and even the presence of soils layers with risk of liquefaction has necessitated the development and application of various ground improvement techniques.

Vibro-replacement usually named as Stone Columns have been used successfully in multiple works from the 60s, aimed to: (1) Strengthen soft soil with high compressibility, (2) Accelerate the consolidation process, (3) Reduce settlements. (4) Increase bearing capacity and (5) Mitigate the risk of liquefaction in soils susceptible to this phenomenon

The Vibro Replacement technique has found increasing acceptance owing to its "flexibility" not only to limit settlements and to ensure stability but also to mitigate liquefaction potential in earthquake prone regions. The range of structures varied from

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highways, railways and airports to marine structures, power plant structures, chemical plants and storage tanks.

The aim of ground improvement design is usually to achieve an acceptable level of settlements and allowable bearing capacity. However, in this area the criterion for the design is the liquefaction mitigation, due to the presence of soil layers with high risk of liquefaction.

Soil liquefaction is reached once the induced seismic shear stresses provoke build up of the excess pore pressure to an extent that leads to the loss of the soil effective stress. As a consequence of soil liquefaction large movements and settlements take place during the earthquake, which can produce important damages in the structures or even the complete collapse.

According to the causes of liquefaction, its mitigation by means of Vibro Stone Columns is achieved due to the incremental benefits of the following positive effects, as has been stated by [1] and [2]:

1. Soil densification and increase of the in-situ lateral stress (increase in CRR).
2. Reinforcement of the soil with the stiffer columns of compacted gravel (reduction of CSR = Cyclic Stress Ratio)
3. Increment of drainage of earthquake-induced excess pore water pressures from the in-situ soils. (reduction of CSR)

Different techniques of soil improvement can be used in seismic areas, but the advantage of Stone Columns bears on simultaneous action on soil densification, on the stress reduction in the soil and on the drainage of the soil to be improved, Besides they can be implemented in all soil types and show their ability to maintain their integrity under the inertial and cinematic direct dynamic effect, with no risk of internal failure of the column because of their granular constitution. In [3] can be seen a simplified method to combine this three effects for designing stone columns against liquefaction.

2. Case Study: Soil Improvement in the Kingston Container Terminal Expansion Project, Jamaica. Liquefaction Mitigation.

The Kingston Container Terminal concession holder has awarded Phase 1 of the Kingston Container Terminal Expansion Project to the consortium formed by the Joint Venture VCGP-EMCC JV (VJV) and Sodraco.

Phase 1 of the project comprises: Dredging of the channel and the berths and refurbishment, reinforcement and upgrade to seismic standards of approximately 1200m quay wall. KELLER as specialist subcontractor is responsible for design, construction, testing and completion of the Soil Improvement Works in order to mitigate the risk of liquefaction during an earthquake in some soil layers under the existing quay.

Table 1. Soil profile for KCT2.

No	Elevation (mCD)	Soil	Ysa (kN/m^3)	c (KN/m^2)	Cu (kN/m^2)	Φ (°)	E (Mpa)
1	+2.80	Compacted silty sand	20	2	-	35	50
2	-0.80	Loose silty sand	17	0	-	29	15
3	-7.00	Dense silty sand	19	0	-	33	40
4	-14.00	MLS : Sandy silt	19	0	105	32	26
5	-30.00	MLC : Clayey silt	20	10	130	32	36

Based on the latest geotechnical site investigations, the soil on the site can be classified as mainly silty and sandy materials. In spite of a relatively large variation along the length of the quay, the soil profile can be roughly split in two parts: the top 12-14 m with different layers consisting of more sandy like materials and the deeper layers consisting of clayey silts, both with varying consistency, traces of organic material and some calcareous inclusions. The described silty sandy materials are typical layers showing a risk of liquefaction.

The loose silty sand and dense silty sand are liquefiable. As part of the works both layers will be improved by the installation of stone columns. For the overall quay stability, improved soil properties based on the project's requirements were considered after treatment: $\Phi = 35°$; E = 22 MPa in the loose silty sand and E = 44 MPa in the dense silty sand.

Six CPTs have been carried out in the initial investigation campaign and were completed by 20 CPTs for the second investigation campaign. In this article, we have chosen CPT 17 showing the highest risk of liquefaction depicted in Figure 1 (predrilling from 0 to 4m depth):

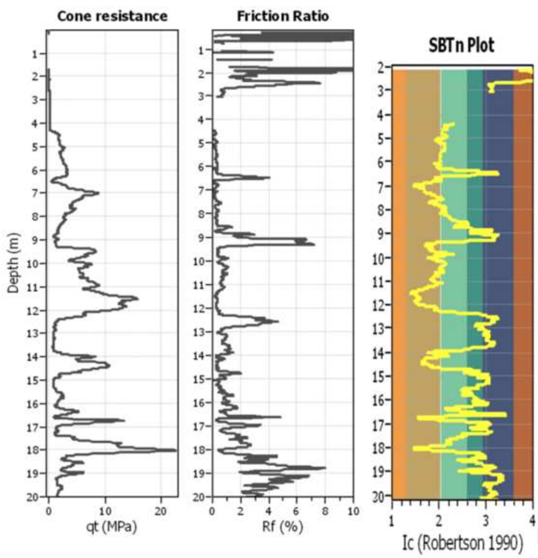

Figure 1. CPT 17 Logs.

For this project, the design earthquake magnitude is about 7, with a Peak Ground Acceleration (PGA) of 0.29g. Then, according to [4], from their tables, the equivalent number of cycles due to earthquake loading is about 12. According to [5], the duration of the earthquake is about 20 seconds.

2.1. Quay Refurbishment Design

Considering the functional requirements and the proposed refurbishment solution, the following major problems of the new structure are the berth deepening, the seismic stability of the structure and the vertical and horizontal bearing capacity of the front wall.

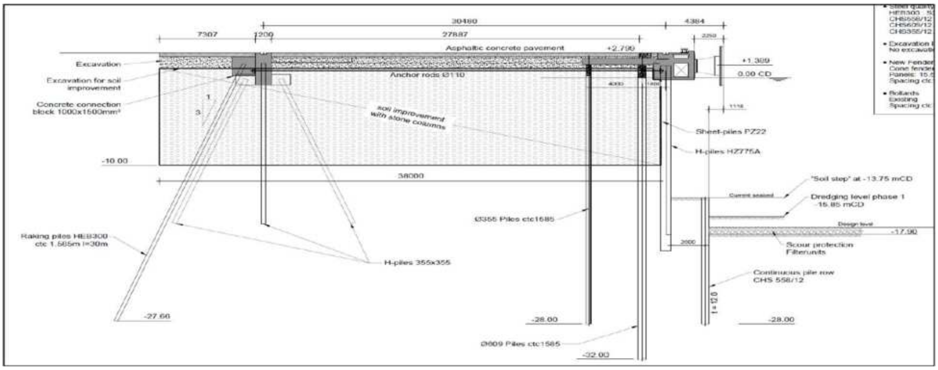

Figure 2. Quay refurbishment solution.

The aim of the project was to improve the soil in both active and passive zones of the combi-wall. This is mainly achieved by installing piles. The improved strength of the soil at both sides of the retaining wall will guarantee horizontal stability in normal condition and also during earthquakes.

Furthermore soil compaction through the installation of stone columns is applied to primarily mitigate the risk of liquefaction. An additional function is that the improved soil properties due to stone column installation reduce the active earth pressure of the system which is beneficial for horizontal stability. It therefore also limits the required number and/or dimensions of piles that are installed in the active zone behind and in the passive zone in front of the combi-wall

Two rows of piles will be installed in the active zone behind the combi-wall. Soil treatment was planned in between the crane beams as well as behind the rear crane beam piles to prevent liquefaction around the retaining wall and the anchor system. Applying a pile wall in the passive zone in front of the combi-wall not only improves the soil but also causes mobilization of deeper soils for passive resistance.

2.2. Soil Improvement Treatment Objectives and Properties

The main objective of soil improvement is to mitigate the risk of liquefaction during a seismic event, referring to the above mentioned design conditions. A minimum factor of safety against liquefaction of 1.25 is taken as the design criteria.

The soil improvement consists in Stone Columns with the following properties: Maximum length 15m, diameter 700 up to 1,000 mm, E_{oed} = 100 MPa, friction angle 42°.

The above mentioned characteristics are the average ones considered for the crushed stone compacted by the M-vibrator designed and manufactured by Keller. The diameter is depending on the local characteristics of the identified layers and whether pre-drilling is used or not.

The design takes into account the vibrocompaction effect in-between the columns, in case of clean sand with a fine content smaller than 15%, as a conservative rule. The maximum average value assumed to be reached in between the stone columns was CPT tip resistance of qc = 6 MPa, i.e. N(60) = 15.

The stone column layout also takes into account the presence of buried diameter 110 mm tie rods, to be preserved 2m beneath pavement.

2.3. Mitigation of Liquefaction Risk. Liquefaction Risk before Treatment

The liquefaction potential Index LPI or IL is defined by [6] and predicts the performance of the whole soil column and the consequence of liquefaction at the ground surface. The following assumptions were made by [6] in formulating index IL. The severity of liquefaction is:

- Proportional to the thickness of the liquefied layer;
- Proportional to the proximity of the liquefied layer to the ground surface;
- Related to the factor of safety (FS) against the initiation of liquefaction but only the soils with FS < 1 contribute to the severity of liquefaction.

Furthermore, the effect of liquefaction at depths greater than 20 m is assumed to be negligible, since no surface effects from liquefaction at such depths have been reported. Iwasaki et al. (1982) proposed the following form for the index IL that reflects the stated assumptions:

- Very low if IL = 0; Low if 0 < IL ≤ 5; High if 5 < IL ≤ 15; Very high if IL > 15.

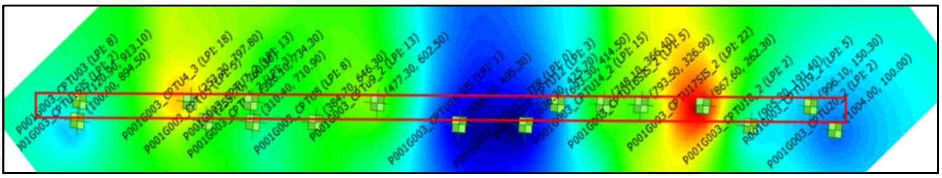

Figure 3. LPI for the latest soil investigation campaign.

Figure 3 shows the location of the highest LPI nearby CPT17 in a range of 20.

2.4. Calculation of Liquefaction Risk after Treatment.

The calculation of the factor of safety FOS after treatment against the liquefaction potential is carried out in a way to take into account:

- The local soil profile: a calculation for each CPT is made with regard to the liquefaction risk.
- The increase of soil compaction due to the stone columns (i.e. increase of CRR/decrease of CSR);
- The accelerated release of the excess pore pressure, achieved in the subsoil during the earthquake, due the drainage effect of the stone columns. The software used for this second calculation step is [7]. This aspect shall not be neglected as it is an important component of the anti-seismic effect of the stone columns.

In order to obtain FOS > 1.25 condition, [8] and [9] suggest that the ratio of average excess pore pressure ratio $r_{u,ay}$ shall be equal to or less than 0.5, while [1], recommend that the maximum excess pore pressure ratio $r_{u,aax}$ shall be equal to or less than 0.6. The excess pore pressure ratio is defined as: $r_u = u/\sigma'_0$ with “u” as excess pore pressure, and σ'_0 as vertical initial effective stress.

2.5. Soil Improvement Due to Stone Columns

Installation of stone columns will have the following improvement effects on the existing soil:

- The improvement due to incorporation of material with a higher and controlled friction angle and higher E-modulus. This allows a reduction of the shear stress ratio CSR;
- Compaction of the surrounding soil by lateral stress and/or vibration during the installation of the stone column. This effect is only marginal in layers with fine content more than 15% (generally not liquefiable) and increasing when the sand is clean.

Baez and Martin [1] developed a method for quick evaluation of the shear stress reduction factor Kg. During the design phase, several stone column grids and diameters are considered, leading to the following area replacement ratio and CSR Reduction Factor Kg calculated after above mentioned publication:

Table 2. Initial and Reduced CSR after Stone Column installation.

SC (m)	SC grid (m2)	Ar (%)	Kg	Min initial CSR	Reduced Min CSR post SC	Max initial CSR	Reduced Max CSR after SC
0.7	7.9	4.9	0.85	0.19	0.16	0.26	0.22
1.0	7.9	10.0	0.75	0.19	0.14	0.26	0.195
0.7	9.0	4.0	0.88	0.19	0.17	0.26	0.23
1.0	9.0	8.1	0.78	0.19	0.15	0.26	0.20

2.6. Release of Excess Pore Pressure

The release of excess pore pressure is calculated according to the finite element methodology proposed by [7], according to the input data detailed in Table 3.

The soil will liquefy under a certain number of cycles. As seen in Figure 4, the equivalent number of cycles due to earthquake loading is about 12. The number of cycles leading to the liquefaction of the soil is determined with the method by [10]. The input data for the method by [7], are detailed below, related to CPT17 with highest liquefaction risk.

Also the figures below show the number of cycles to liquefaction Nl derived from [10].

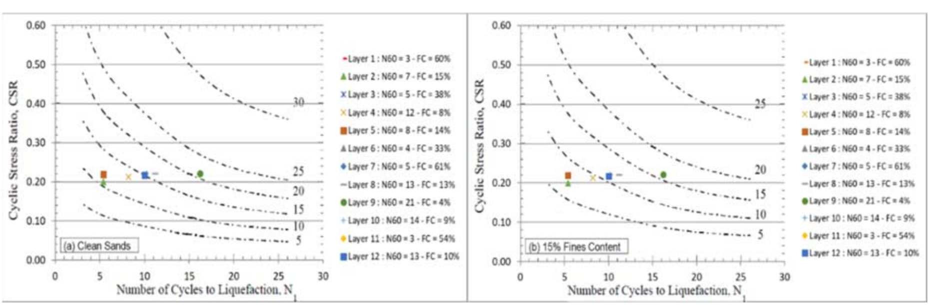

Figure 4. CPT17 input parameters with a SC diameter of 0.7m and 1.0m.

Table 3. Iniput Soil Parameters required.

Layer	Depth (m)	N1 (60)	CSR	CSR reduced (Ø0.7m)	CSR reduced (Ø0.7m)	FC (%)	Kv (m/s)	kh/kv	N_L (Ø0.7m)/ N_L (Ø1.0m)	Initial FoS
1	2.0-4.9	3	0.20	0.17	0.15	60	1.0E-08	20	50/50	2.0
2	4.9-6.2	12	0.23	0.20	0.18	15	4.5E-06	2	5/7	0.3
3	6.2-6.6	5	0.24	0.21	0.18	38	6.2E-09	10	50/50	2.0
4	6.6-7.4	12	0.25	0.21	0.19	8	1.4E-05	1	8/10	0.5
5	7.4-8.3	12	0.26	0.22	0.19	14	5.3E-06	2	5/7	0.3
6	8.3-8.8	5	0.26	0.22	0.20	36	3.1E-07	5	50/50	2.0
7	8.8-9.4	4	0.26	0.22	0.20	61	3.5E-09	20	50/50	2.0
8	9.4-11.2	13	0.26	0.22	0.20	13	3.4E-06	2	11/13	0.6
9	11.2-12.0	21	0.26	0.22	0.20	4	7.6E-05	1	16/20	1.0
10	12.0-12.3	14	0.26	0.22	0.20	9	2.0E-05	1	10/13	0.6
11	12.3-14.0	3	0.26	0.22	0.20	54	5.0E-09	20	50/50	2.0
12	14.0-14.7	13	0.26	0.22	0.19	10	2.0E-05	1	10/12	0.6

Figures 5 and 6 show the results of the excess pore pressure ratio after 20 seconds of earthquake for the reference test CPT17 and two different hypothesis according to the stone columns diameters:

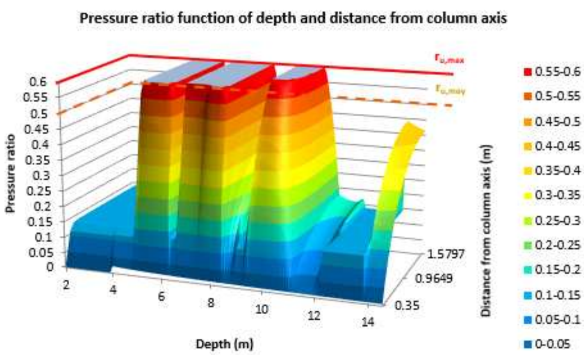

Figure 5. CPT 17: Rectangular grid 3.17x2.5 m Ø 0.70 m.

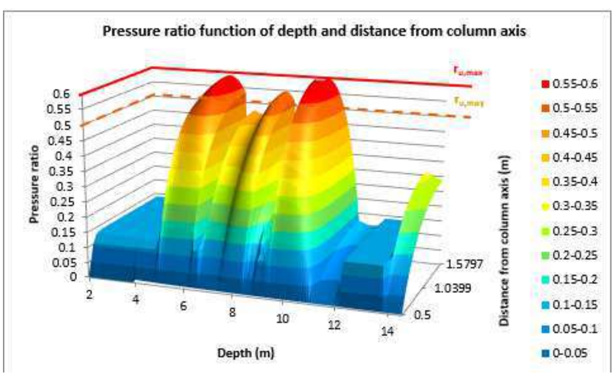

Figure 6. CPT 17: Rectangular grid 3.17x2.5 m Ø 1,0 m.

The results (Figure 5) show $r_{u,max}$ and $r_{u,moy}$ above 0.6 and 0.5 respectively, i.e. FOS > 1.25, for some layers in the case of diameters of 0.70 m. The calculation is then run again (Figure 6) with the same grid size, considering predrilling for enlargement of the diameter up to 1.0m between -5.0 and -11.0 depth. It mitigates the risk of liquefaction with a factor of safety superior to 1.25.

The liquefaction mitigation for the CPT17 soil conditions, the advantage of Vibro-replacement bears on their simultaneous action takes place into account the following variations of the stone column diameter with the depth (see Figure 2):

- From +2.80CD to -8.2CD: Predrilling; Stone column diameter between 0.8m a nd 1.0m from FGL to -2.0 CD; Ø 1.00m from -2.0 to -8.2CD;
- From -8.2 CD to -11.7 CD: Minimum stone column diameter of 0.70m.

The design treatment grid will be a rectangular grid of 3.17x2.5m.

2.7. Trial Test

Based on the CPT17, the stone columns in KCT2 were installed in a rectangular grid of 2.5x3.17m, with a diameter of 1.0m from 2.8CD to -8.2CD and a diameter of 0.7m from -8.2CD to -11.7CD (see Figure 2).

A field test was carried out in the area of CPT17 in order to confirm the design. The test has shown that the vibrocompaction effect worked properly in the clean sand layers while its efficiency decreases with the increasing fines content, and especially when the Ic coefficient, according to [1], exceeds values of 2.2, i.e. approximately for 20% of fines content.

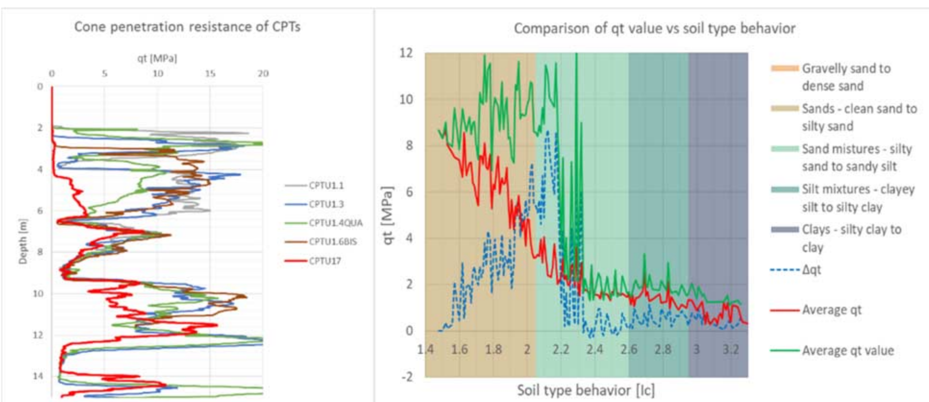

Figure 7. Cone penetration resistance before and after treatment.

Nevertheless, the expected increase of cone penetration resistance is limited and especially with Ic between 2.2 and 2.6, i.e. for liquefiable silty soils, there was no necessary improvement. As a result, the stone columns grid was slightly densified to a mesh of 2.35x3.17m.

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