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ORIGINAL PAPER

Vibro-compaction trial for soil improvement in the northwest of Abu Dhabi, UAE

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Abstract

This case study evaluated the liquefaction risk and application of vibro-compaction for soil improvement in a construction project site on Saadiyat Island, Abu Dhabi, UAE. Abu Dhabi is designated as Zone 0 according to the Uniform Building Code (UBC-97), and we discuss the design criteria for vibro-compaction that were adopted to mitigate the liquefaction risk, the trials conducted to establish the application criteria for vibro-compaction, and the general practices related to vibro-compaction. Specific studies conducted in Abu Dhabi Emirate indicate that the seismicity in the region is low, and the probability of liquefaction is very limited. However, during the pre-project soil investigation phase, the analysis of potential soil liquefaction indicated that certain examined areas have the potential to undergo liquefaction. The liquefaction potential was assessed based on a combination of safety factors obtained for an earthquake with a magnitude Mw = 6 and the corrected cone tip resistance (qc). The acceptability criteria for improved soil are based on cone penetration test (CPT) results. The target qc is accepted as 2.7 MPa and the treatment depth would be a minimum of − 3.5 m. A field trial was conducted to determine the optimal compaction grid spacing that meets the specified acceptance criteria and aligns with the project's design criteria. Based on the trial evaluation, a vibro-compaction grid spacing of 4 × 4 m was appropriate. Consequently, a total of 4125 points at the construction site underwent vibro-compaction.

Keywords Vibro-compaction · Liquefaction · Soil improvement · Reclamation · CPT

Introduction

Soil improvement is a wide-reaching subject that enhances soil properties through tailored physical, chemical, or biological methods for specific engineering applications and local soil conditions, often requiring specialized construction techniques for effective implementation. One such well-established soil improvement method is vibro-compaction. Vibro-compaction, the oldest dynamic deep compaction method, is a practical technique to enhance the bearing capacity and settlement characteristics of weak soils. This method, which originated in 1936 through the efforts of the Johann Keller Company, has delivered impressive results in compacting cohesionless soils (Kirsch and Bell 2012).

Vibro-compaction generates lateral vibrations and vibratory forces, with a critical acceleration threshold at approximately 0.5 g, leading to destruction in granular soils. Acceleration exceeding 1.5 g reduces shear strength, causing soil liquefaction, while further acceleration above 3.0 g leads to soil dilation (Han 2015). Vibro-compaction typically yields positive results when applied to soils composed primarily of sand with a content of approximately 10% or less fine-grained material passing through a #200 sieve (Martin and Lew 1999).

Cone penetration test (CPT) data are widely used to evaluate the effectiveness of vibro-compaction. CPT values, including cone resistance, sleeve friction, friction ratio, and pore water pressure, are compared before and after vibro-compaction to evaluate changes in soil density. An increase

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in coarse gravel and gravel content in the soil can raise permeability, resulting in a significant water loss with $k \approx 10-2$ m/s. This may hinder the depth vibrator's penetration, preventing it from reaching the necessary depth in some or all areas (Kirsch and Kirsch 2017). Although cohesive soils are more resistant to liquefaction than clean sands, liquefaction also occurs under relatively long-term and high-intensity seismic action (Moseley and Kirsch 2004).

Investigation has focused on several projects where the vibro-compaction method has been used to address liquefaction risk, settlement issues, and ground reinforcement. Marwan and Abdelaziz (2015) conducted a study on soil improvement for the reclaimed granular fill on Pearl Jumeira Island, enhancing strength, and stiffness, and to prevent liquefaction during seismic events. They used vibro-compaction for deep soil improvement and vibratory roller compaction for shallow improvement to meet project criteria. Performance was evaluated using CPT, in situ density measurements, and settlement monitoring, all aligning with design assumptions. The risk of liquefaction was successfully eliminated, resulting in a safety factor exceeding 1.25. Previous studies covering similar projects provided guidance for this study. Ramanathan and Prasad (2019) examined soil characterization, performance criteria, and soil improvement measures for liquefaction reduction in a case study of a corporate campus in northern India. This campus comprised 15 structures, including academic and residential buildings, designed for open/pile foundations. The structures varied from one to nine storeys, with a maximum pressure of 150 kPa. The ground comprised loose to medium dense sandy soil, with a fine percent of under 12. This area was situated in a seismic zone with a peak ground acceleration of 0.24 g. Soil deposits up to 10–12 m deep were susceptible to liquefaction during earthquakes. Annam and Raju (2015) demonstrated the effectiveness of vibro-compaction for soil improvement in a school construction project in Noida, Uttar Pradesh, India. The site had loose to medium dense fine sand up to 9 m deep, which was susceptible to liquefaction due to its location in Seismic Zone IV. Vibro-compaction was used to mitigate liquefaction and enhance bearing capacity. Post-treatment Standard Penetration Test (SPT) N values exceeded the required criteria, and settlements during a field plate load test remained within acceptable limits.

The present case study aimed to share field trial experiences of vibro-compaction application as a soil improvement method. It used pre-application CPT results for liquefaction analysis, determination of application design, realization of design assumptions, and identification of the range of vibro-compaction applications. The obtained experiences are intended to provide guidance for soil-related issues in similar soil conditions and geological environments.

Regional geology

The study area is located on Saadiyat Island in Abu Dhabi. The island area is approximately 27 km^2 and is located to the north of the main island on which the existing Abu Dhabi city is mostly concentrated. The project site is a reclaimed coastline that extends about 500 m from Saadiyat Island's shoreline, covering an area of approximately 500 m by 500 m. The location of Saadiyat Island, including the study area, is shown in Fig. 1.

The geological features of the UAE and the Arabian Gulf region have been shaped by the deposition of marine sediments, a consequence of recent fluctuations in sea levels. Apart from the mountainous regions in the northeast, where the UAE borders Oman, the country primarily consists of

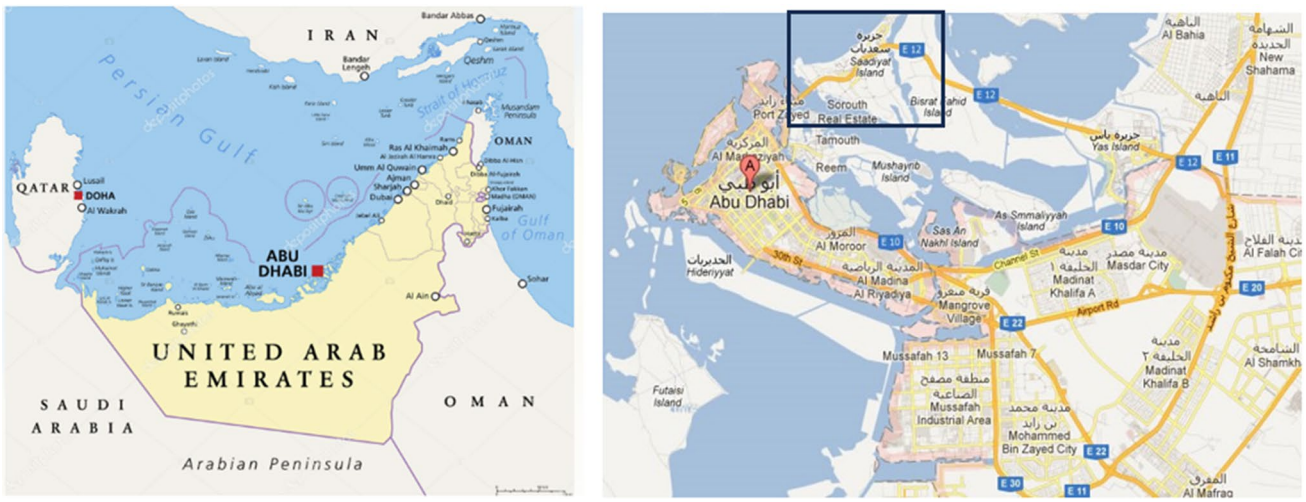


Fig. 1 Vibro-compaction trial site

low-lying areas characterized by Quaternary to Late Pleistocene deposits (Poulos 2018). Abu Dhabi's landscape is characterized by a straight coastline with historical channels and bays. Coastal areas are covered by marine sand and silt sediments. Wind erosion, capillary action, and evaporation have created extensive sabkha deposits, particularly concentrated around the bays, forming hypersaline accumulations. The deposits on this surface cover alternating layers of calcarenite, carbonate sandstone, gypsum, sand, and cemented sands (Fugro ME 2008).

Saadiyat Island is a natural island which has been extensively modified by recent and past land reclamation activities. Recent reclamation has been undertaken by constructing a temporary containment bund with rock armoring, then backfilling using dredged material from the sea bottom placed hydraulically through pipeline discharge. The current formation level of the hydraulic fill is approximately at elevation + 1.5 m. Based on water levels encountered during exploratory drilling and monitoring of standpipe piezometers installed following drilling, groundwater levels between − 0.5 m and + 0.5 m are apparent at the site.

Geotechnical profile

In general, boreholes revealed granular deposits of fill material comprising of medium dense, slightly silty, fine to coarse-grained sand from + 1.50 m to + 0.50 m, a thickness of 1.0 m layer of loose, slightly silty to silty sand, underlies the previous layer and extends to − 2.50 m. Below this, there is a stratum of dense to very dense sand with slight to moderate silt content, extending to a depth of − 5.50 m. The top of a rock layer was encountered at an approximate depth of − 5.50 m and extended to the full depth of the boreholes depth of − 78.50 m. The bedrock consisted of very weak to weak sandstone/calcarenite, underlain by alternating very weak to weak siltstone, mudstone, and gypsum (Fugro 2010).

Materials and methods

The aim of the soil improvement by means of vibro-compaction is to improve the ground in line with the findings/conclusion of the soil investigation report carried out for the project area. The scope of the Ground Improvement works by vibro-compaction includes mitigating liquefaction potential for UBC Zone 0 to eliminate the risk of soil liquefaction during an earthquake with a peak ground acceleration of $a = 0.20$ g at Magnitude 6.

Before the start of the vibro-compaction work, a field trial was performed in the project area. The purpose of these trials was to establish parameters for the most feasible grid spacing of compaction points that meet the design criteria laid down for this project. The sequence of vibro-compaction works involved the following steps: The pre-CPT Test was carried out at the trial area, encompassing three different grids with an equilateral triangular spacing of 3.50 m, 4.0 m, and 4.50 m. Then, probe locations were set out according to the selected grid for the specific area. The installation of vibro-compaction probes was conducted. Post-treatment CPT tests were then conducted to the top of the rock-layer (equidistant from adjacent compaction probes) at a frequency of 1:1000 m^2 to confirm the acceptability criteria over the soil improvement area. Finally, a post-compaction level survey was carried out.

Liquefaction assessment

Liquefaction analyses were carried out based on CPT data carried out as part of the soil investigation works performed in the project area. The general mechanism for earthquake-related soil liquefaction is as follows. With the propagation of waves generated by an earthquake, cyclic shear stresses cause volumetric compression in loose, saturated, cohesionless soil. This compression forces soil particles to become closer and denser, facilitating the transfer of load from one particle to another. This load transfer increases the pore water pressure. Under the sudden and rapid effect of seismic shaking, the soil is subjected to undrained loading, leading to an increase in excess pore water pressures. As the pore water pressures increase, effective stress diminishes. This process is referred to as liquefaction. Liquefied soil no longer behaves like a solid but instead behaves like a liquid, losing its strength (Marcuson 1978). Loose or unconsolidated soils undergo compression during liquefaction and re-solidification processes, causing the soil to settle. Concurrently, sand boils may reach the surface as excess pore water pressures are relieved. Determining the liquefaction potential necessitates the calculation or estimation of two variables based on CPT analysis results to determine the liquefaction resistance of soils. These are the cyclic shear stress ratio (CSR) representing the seismic demand in the soil layer and the cyclic resistance ratio (CRR) indicating the soil's resistance to liquefaction. Seed and Idriss (1971) developed Eq. 1 for the calculation of the CSR.

$$\mathrm{CSR} = (\tau_{av}/\sigma'_{vo}) = 0.65(a_{max}/g)\,(\sigma_{vo}/\sigma'_{vo})\,r_d \quad (1)$$

In Eq. 1 a_{max} is the maximum horizontal ground acceleration created by the earthquake and acting on the surface, σ_{vo} is the total vertical stress, σ'_{vo} is the effective vertical stress, (g) is the gravitational acceleration, τ_{av} is the average cyclic tensile strength, and r_d is the stress reduction coefficient.

The coefficient for effective stress reduction can be calculated using the method described in Eq. 2a including the

mean and range of values recommended by Seed and Idriss (1971).

$$r_d = 1.0 - 0.00765z \text{ for } z \leq 9.15 \text{ m} \quad (2a)$$

$$r_d = 1.174 - 0.0267z \text{ for } 9.15 \text{ m} < z \leq 23 \text{ m} \quad (2b)$$

where z is the depth below the ground surface in meters.

Recent field performance data have shown that the existing CPT-based correlation by Robertson and Campanella (1985) for clean sands is generally good. Based on discussions at the 1996 NCEER workshop (NCEER 1997), the curve by Robertson and Campanella (1985) has been adjusted slightly at the lower end. The resulting recommended CPT correlation for clean sand is shown in Fig. 2.

Studies have confirmed the criteria shown in Fig. 2 are conservative. Robertson and Wride (1998) confirmed these criteria by comparing with SPT and other data from the site they studied. Gilstrap and Youd (1998) also found that these criteria correctly predicted liquefaction with > 85% reliability at 19 sites. Equations 3 and 4 in Robertson and Wride (1998) approximate the clean-sand curve in Fig. 2.

$$\text{If } (_{qc1N})_{cs} < 50 \text{ CRR}_{7.5} = 0.833[(q_{c1N})cs/1,000] + 0.05 \quad (3)$$

$$\text{If } 50 \leq (q_{c1N})_{cs} < 160 \text{ CRR}_{7.5} = 93[(q_{c1N})_{cs}/1,000]^3 + 0.08 \quad (4)$$

Where $(q_{c1N})_{cs}$ = Clean-sand CPT resistance normalized to about 100 kPa.

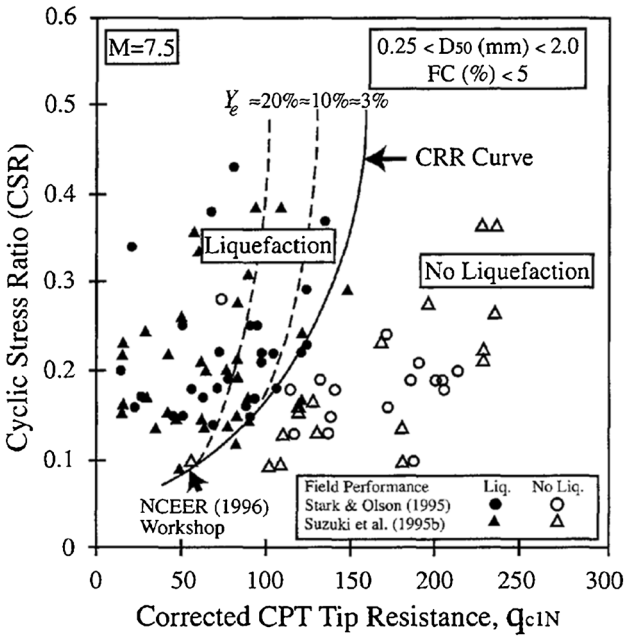


Fig. 2 Curves recommended for calculation of CRR from CPT Data (NCEER 1997)

In this method, some adjustments are necessary when using CPT tip resistance (qc) in determining liquefaction resistance. Normalization of qc tip resistance may be performed using Eqs. 5and ′ 6.

$$q_{c1N} = C_Q(q_c/P_a) \quad (5)$$

$$C_Q = (P_a/\sigma'_{vo})^n \quad (6)$$

Where C_Q represents the normalizing factor for cone penetration resistance.

In this context, (C_Q) is the correction factor for cone penetration resistance, (Pa) represents a reference stress of 100 kPa, (n) varies with soil type, (qc) is the measured tip resistance, and (qcN1) is the dimensionless corrected tip resistance with respect to the upper layer stress. Values of C_Q exceeding 1.7 are not considered in practice due to low effective stress at shallow depths. The constant "n" falls between 0.5 and 1.0, depending on the soil's grain characteristics (Olsen 1997).

The soil behavior type index (I_c) used to define the soil type was calculated from Eqs. 7, 8, and 9.

$$I_c = [(3.47 - \log Q)^2 + (1.22 + \log F)^2]^{0.5} \quad (7)$$

$$Q = [(q_c - \sigma_{vo})/P_a][(P_a/\sigma'_{vo})^n] \quad (8)$$

$$F = [f_s/(q_c - \sigma_{vo})] \times 100\% \quad (9)$$

Where Q represents the normalized tip resistance and F is the normalized friction ratio.

Figure 3 was created with an exponent (n) set to 1.0, an appropriate choice for clay soil types. In contrast, clean sands are better represented by an exponent of 0.5, while an intermediate exponent value (between 0.5 and 1.0) is recommended for silts and sandy silts (Robertson and Wride 1998).

Robertson and Wride (1998) proposed a method for calculating the I_c by differentiating clay soils from sand and silt soils. Assuming an exponent of 1.0 for clays, the use of Eq. ″ 10 will calculate the dimensionless CPT tip resistance (Q) to classify soil behavior based on CPT response.

$$Q = [(q_c - \sigma_{vo})/P_a][P_a/\sigma'_{vo}]^{1.0} = [((q_c - \sigma_{vo})/\sigma'_{vo}] \quad (10)$$

Equation 11 adjusts the normalized penetration resistance (qc1N) for silty sands to an equivalent value in clean sand $(q_{c1N})_{cs.}$

$$(q_{c1N})_{cs} = K_c q_{c1N} \quad (11)$$

K_c represents the correction factor accounting for grain characteristics and is determined through Eq. 12a and 12b, as proposed by Robertson and Wride (1998).

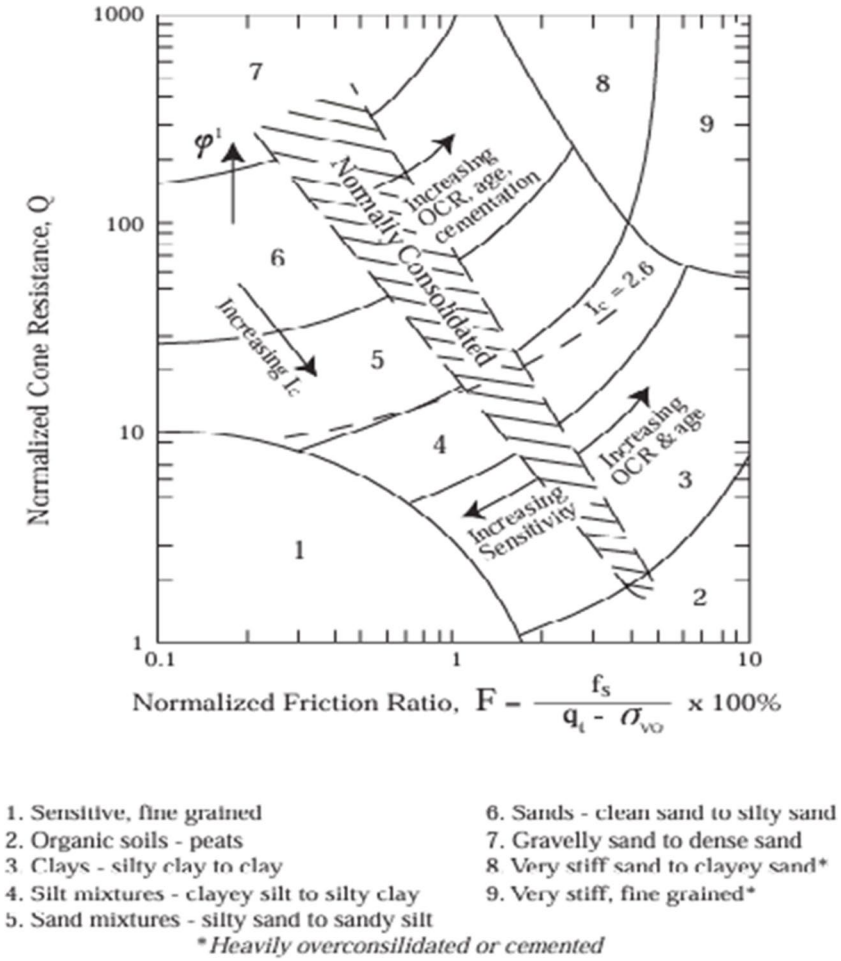


Fig. 3 CPT-based soil behavior-type chart (Robertson 1990)

$$\text{for } I_c \leq 1.64 \; K_c = 1.0 \tag{12a}$$

$$\text{for } I_c > 1.64 \; K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88 \tag{12b}$$

To determine CRR for different earthquake magnitudes, one can utilize Eqs. (3) and (4) by selecting suitable values for Ic and Kc. Once CRR7.5 is calculated, it can be adapted to other magnitudes by multiplying it with a suitable scaling factor specific to the desired magnitude. The ratio of CSR, which characterizes the seismic strength, to CRR, which characterizes the soil resistance, gives the safety factor (SF = CSR/CRR) against liquefaction of that soil. If the safety factor is less than 1, it indicates that the region is at risk of liquefaction. If the safety factor is greater than 1, it indicates that the region is not at risk of liquefaction (Youd et al. 2001).

The soil investigation studies were conducted by including all building and foundation design parameters, assumptions, and tolerances, and the liquefaction risks of the soil were also investigated. In this regard, a total of 42 CPT tests were conducted in the field. The piezocone penetration tests were conducted to refusal in accordance with code of practice for site investigations (BS 5930 1999), using a 20-tonne hydraulic-powered, lorry-mounted, penetrometer. The testing was performed using an electrical piezocone, with a projected area of 15 cm^2, an apex angle of 60° and a friction sleeve area of 200 cm^2. The cone contained electrical load transducers measuring the cone tip resistance, sleeve friction, and pore pressure. Readings of the cone tip resistance, sleeve friction resistance, and pore pressure are obtained continuously via an electric cable in the cone rods and data is transferred to a computer. During the test, the data is displayed on the computer screen and continuously recorded to the hard drive for later processing. The results of the PCPTs were presented in the form of plots of cone resistance, sleeve friction, friction ratio, and pore pressure versus depth of penetration. A liquefaction sample calculation performed based on the CPT results (Youd et al. 2001) for the soils of the study area is presented in Table 1.

Liquefaction potential was assessed based on a combination of safety factors returned for a Mw = 6 earthquake and corrected cone tip resistance qc. All CPT plots combined are presented in Fig. 4, the average qc is taken to be at 1.5 MPa at a depth of 1.70 m below ground level (m bgl) in which the safety factor returned as 0.25 (safety factor above 1.25: No liquefaction, safety factor between 1 and above 1.25 Marginal, safety factor below 1: liquefaction).

After the analysis, it was concluded that some checked areas have the potential to liquefy. Low values of qc of 0 to 3 MPa were present from 1.70 to 4.20 m bgl. In the above analysis, if the identified liquefiable depth zones were not excavated out, it was recommended to enhance the soil resistance against liquefaction through soil improvement methods for both shallow and piled foundations.

Brief design base for soil improvement by vibro-compaction

The design of soil improvement is carried out for parameters to mitigate liquefaction potential for The Uniform Building Code (UBC-97) Zone 0 and eliminate the risk of soil liquefaction during a hypothetical earthquake that would cause a peak ground acceleration in the soil of $a_{max} = 0.20$ g at magnitude Mw = 6. Based on UBC-97, Abu Dhabi is in Zone 0 seismicity and in specific studies carried out in Abu

Table 1 Calculation example for the study area (Fugro 2010)

CPT ID	Depth m	qc MPa	CSR_{peak} kPa	CRR7.5 kPa	Fs	Observations for Mw = 6
Combined All	1.7	1.5	0.292	0.108	0.25	Liquefaction

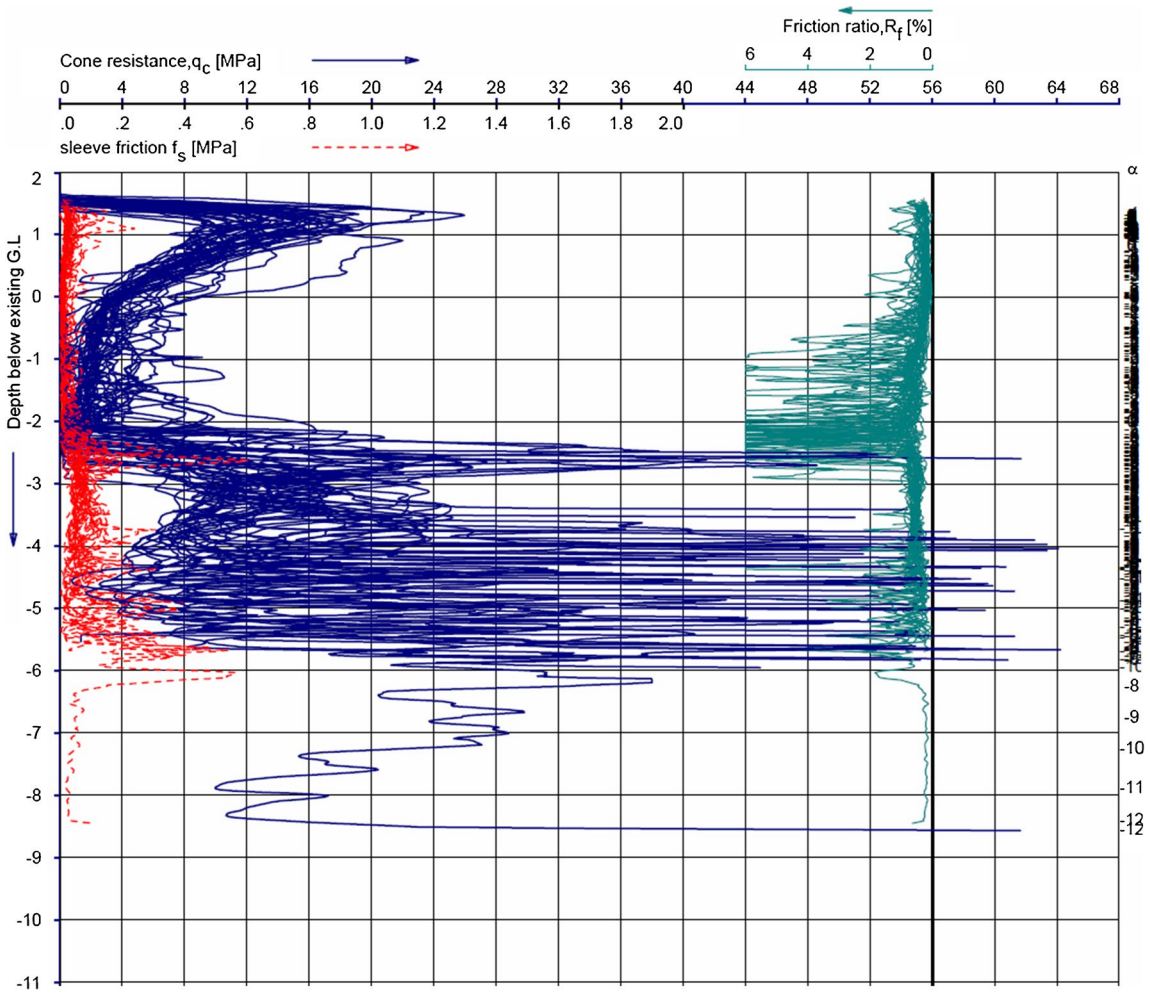


Fig. 4 CPT results consolidated in a single evaluation diagram (Fugro 2010)

Dhabi, the seismicity is considered low, thus the seismic parameters (a_{max} and Mw) used in the liquefaction analyses were determined based on the UBC-97 seismic criteria. The acceptability criteria for improved ground, based on required CPT "qc" values to mitigate potential liquefaction, are based on post-CPT results. The target cone resistance is calculated as qc = 2.7 MPa and the treatment depth shall be a minimum of − 3.5 m. To confirm the adequacy of target cone resistance, liquefaction analysis using the LiquefyPro 3.1B software (CivilTech 2002) was carried out (Fig. 5). The friction ratio was considered 1% for this calculation. The calculation indicates a qc value of 2.70 MPa was adequate down to a depth of 3 m and 3 MPa thereafter down to a depth of 5 m.

The sequence of vibro-compaction work

Vibro-compaction works were planned to include carrying out a field trial involving three different grids of 3.50 m, 4.0 m, and 4.50 m equilateral triangular spacing involving pre- and post-CPT tests. Probe locations were determined according to selected grid for the specific area, and this was followed by pre-CPT testing. Then, vibro-compaction was performed and post-treatment CPT was performed down to the top of the rock-layer (equidistance from adjacent compaction probes) at a frequency of 1:1000 m^2 in order to confirm the acceptability criteria over the soil improvement area indicated, carrying out post-compaction level survey. Finally, buildup of the subsided ground-level (post-vibro-compaction) to the hand-over-level was undertaken with layers of 30 cm and roller-compacted to 95% proctor density.

Assessment of soil improvement work

The assessment of the vibro-compaction works was done with reference to the target CPT design curves and has undergone a staged screening process described below, leading to either acceptance, other corrective or alterative measures. When the average post-treatment CPT-values meet or exceed the applicable qc-target-curve which was 2.7 MPa (silica/calcareous sand), this confirmed the mitigation of liquefaction potential of the treated ground. When the average post-treatment CPT values fall below the required qc-target-curve (layers > 1 m), then further analysis is required. The cone resistance values are disregarded when they fall short of 1 MPa and the corresponding friction ratios exceed 0.5%. Such layers are likely to contain fines much greater than 10% and may be silt or clay, which are not compactable by vibro-compaction (Fig. 6) and no further action is required with respect to soil liquefaction. However, depending on the occurrence of such layer in depth, further analysis involving repeated CPT testing, load testing, and borehole sampling is proposed

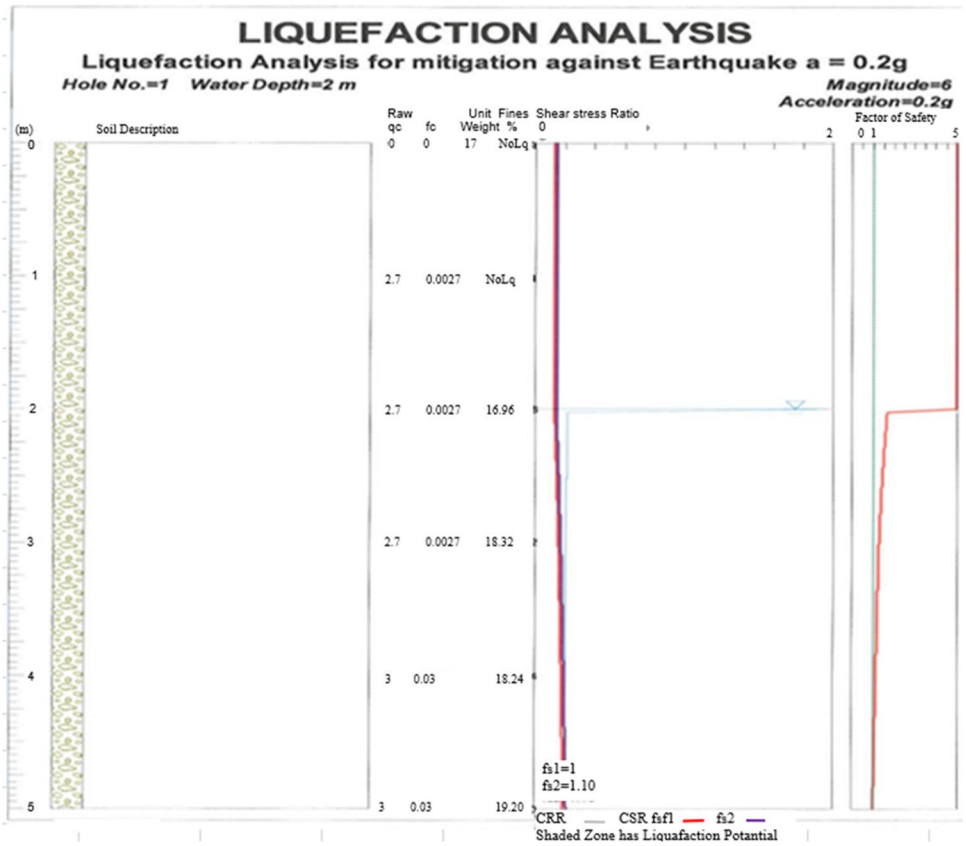


Fig. 5 Liquefaction analysis verifying target cone resistance using 1% friction ratio (HSSG-Keller DR 2010)

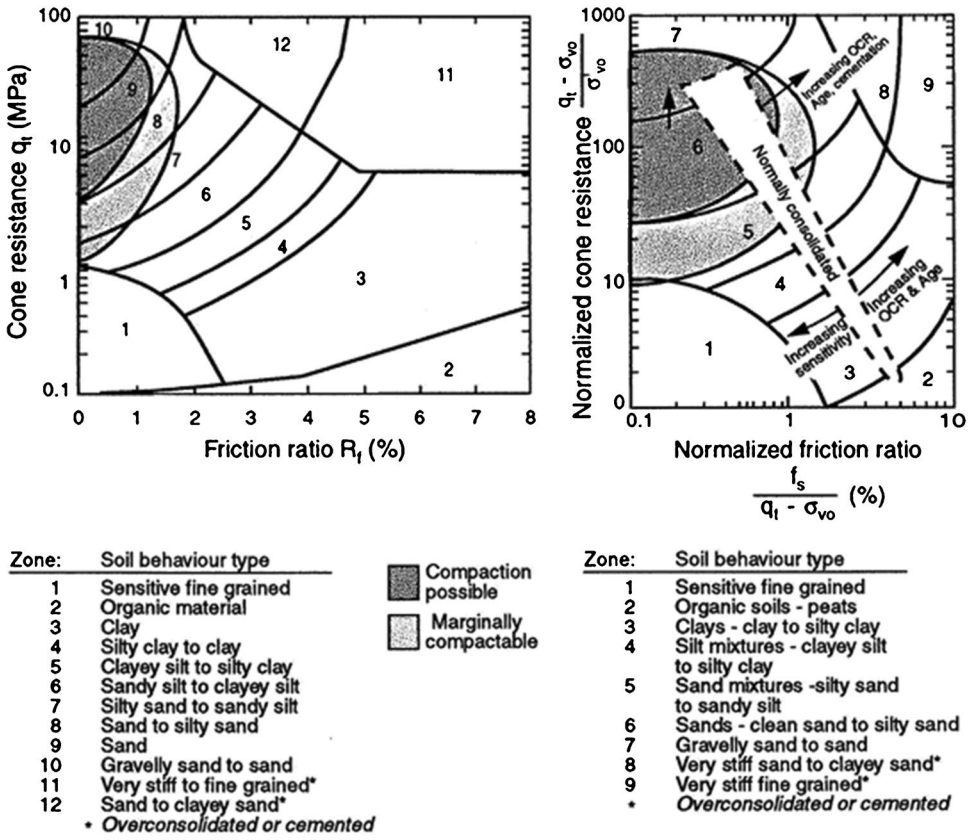


Fig. 6 Classes of soils defined by cone resistance versus friction ratio of CPT method (Lunne et al. 1997)

to establish whether the existence of such a layer does or does not fulfill the objectives of soil improvement and that it would need to be treated by an alternative method. If the cone resistance falls below the target line despite fines being less than 10%, then an appropriate reduction of the adopted spacing is indicated, because of possible change in soil character and existing state of relative density. A repeated trial in such cases is required to derive the optimum spacing. If quality records indicate any deviation from the defined procedure, then a repeat of compaction works adopting the same grid may be necessary.

Vibro-compaction field trial

A vibro-compaction field trial is carried out to determine the suitable compaction grid for achieving the acceptance criteria. The purpose of these trials was to establish parameters for the most feasible grid spacing of compaction points that meets the design criteria for the project. The field trial activities included the installation of compaction points and CPT testing and the selection of the optimum grid spacing corresponding to the site conditions. Figure 7 shows the location of field trials and trial grid layouts. The proposed grid

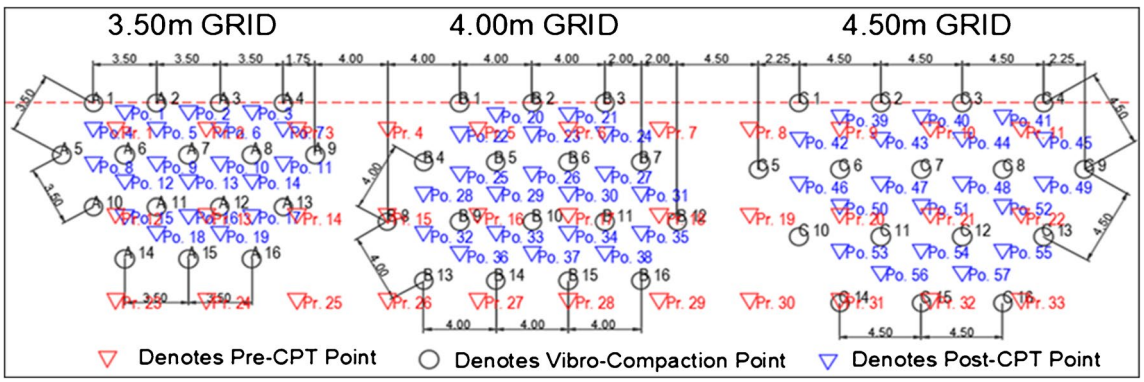


Fig. 7 Vibro-compaction field trials layout

spacing of 3.50 m, 4.00 m, and 4.50 m in a triangular pattern were used. The radius of influence of vibro-compaction in sand depends mainly on the depth vibrator's characteristics and its operation mode, usually ranging between 2 and 4 m. The necessary spacing is based on the chosen machine and is usually determined by experience and field trials, especially for larger projects (Kirsch and Kirsch 2017). A brief scope of the trial works is given in Table 2.

Pre-treatment cone penetration tests (Pre-CPT)

Prior to vibro-compaction trial works, the ground levels of the trial area are recorded at a frequency of 1 reading per 900 m^2 (30 m × 30 m grid), and pre-treatment CPT tests are conducted down to the top of the rock layer, or until refusal, at a frequency of 1 test per 1000 m^2. The pre-CPT points are shown in Fig. 8.

This adopted CPT method complies with BS 1377-9 (1990) Test 3.1 and ISSMFE (1985). The tests were terminated upon refusal which was defined as 40 MPa resistance (qc) (no possible advance). The test results were presented in standard format as recommended by ISSMEF-international reference test procedure. The graphic presentation result includes measured and derived parameters, which are a plot of cone resistance (qc, MPa) versus depth, plot of sleeve friction (fs, kPa) versus depth, plot of friction ratio (Rf, %) versus depth, and plot of pore water pressure (U2-U0 (kPa) versus depth.

Figures 9, 10, and 11 show the results of pre-CPT 13, 5, and 21, respectively. Based on the CPT testing results, soil conditions were observed to be from the ground surface to 1.50 m (1.50 to 0.00 m level) medium dense sand; from 1.50 m to 4.00 m (0.00 m to − 2.50 m level) loose to medium dense sand interbedded with thin layers of very silty/clayey material varying between (0.10 – 0.50 m) and after 4 m (− 2.50 m level) as dense to very dense sand down to refusal depth. The refusal was achieved between depths 5.60 to 7.0 m (− 4.1 to – 5.50 m level).

Trial vibro-compaction work

The upper sandy soils encountered at this site were able to be compacted by vibro-compaction to achieve the target cone resistance of 2.70 MPa barring the thin lenses of silty/clayey layer. Vibro-compaction uses a Keller-made "depth vibrator," a tube with eccentric weights driven by an electric motor. It connects to a power source, has a water injection system, and extends with tubes to reach the desired treatment depth. A 120-ton crawler crane supports the entire setup. To maximize the daily production these vibrators are mostly used as twin-system, i.e., two of the vibrators are suspended from a traverse-beam (a safe load capacity of 25 tons) held by the crawler crane. The data of the Keller deep twin S-700 vibrator was used, and its technical details are as follows: length 3.00 – 4.30 m, diameter 400 – 470 mm, weight 2.45 – 4.20 t, power 120 kW, operation frequency 20 – 40 Hz, and maximum energy 3.0 – 6.0 kNm. A photo of the deployed S-700 vibrator is shown in Fig. 12, and a schematic figure of the vibro-compaction technique is given in Fig. 13. The depression around the vibrator is continuously filled with granular material from the existing ground surface using a shovel loader.

As water is required throughout the vibro-compaction works, a sufficient source within close distance to the working area is needed. The establishment, operation, and maintenance of the water supply to service the vibro-compaction activities are as shown in the sketch given in Fig. 14.

The compaction probe locations were set out on the ground using steel pins 50 cm long. Each pin was clearly marked in accordance with the reference number given in the setting out drawings. After positioning the vibrator at the probe's demarcation, the power and water supply were activated and then lowered into the ground. The penetration is assisted by the vibrations and the high pressure. During the penetration phase, the vibrator's frequency was switched to 70 Hz (penetration mode). Water jetting was cut off at about refusal depth. During the vibro-compaction process, the cone-shaped depression around the vibrator and its extension tubes are continuously filled with the granular material from the existing ground surface using a shovel loader (HSSG-Keller MS 2010). On reaching the desired depth, the vibrator was switched to compaction mode, i.e., 30 Hz frequency. The vibrator was held at that depth for 40 s or 420 amps, whichever occurred first. In the trials, a 100 cm lift was decided upon. After each 100 cm lift, the vibrator was held for 40 s or 420 amps whichever occurred first. The crater formed, was simultaneously in-filled with the surrounding soil. The process of compaction is continued in 1 m lifts up to the ground level. The vibro-compaction points in the trial area are shown in Fig. 15. The aim is to keep water levels unchanged in comparison to the compaction works for pre- and post-CPT tests, if possible.

The vibro-compaction process is continuously monitored in real time using an automatic digital data recorder. This device maintains a continuous graphical record of the installation procedure on a computer. The information provided includes depth versus time plot, power consumption

Table 2 Grid spacing of 3.50 m, 4.00 m, and 4.50 m (HSSG-Keller FTR 2010)

Grid	Spacing (m)	Mean depth (m)	Probe count
Triangular	3.50	7.030	16
Triangular	4.00	6.86	16
Triangular	4.50	6.9	16

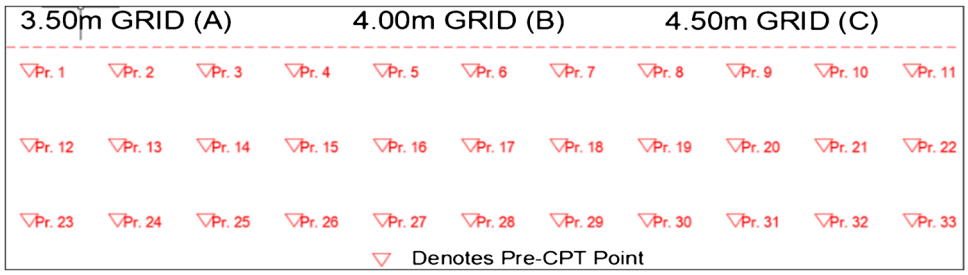


Fig. 8 Pre-CPT points in the trail area with grid spacing of 3.50 m, 4.00 m, and 4.50 m

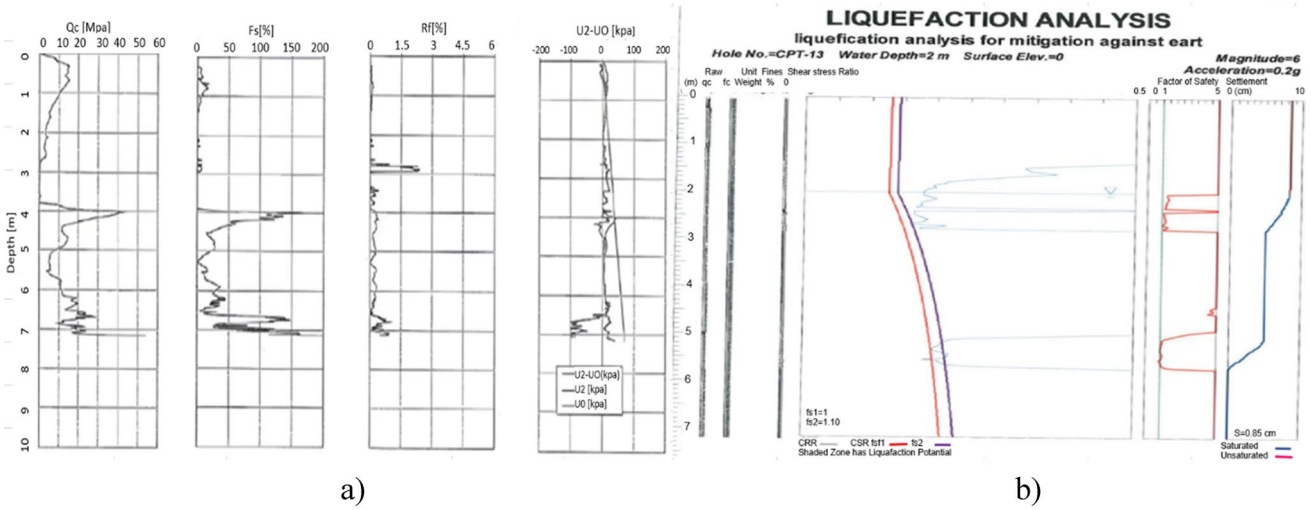


Fig. 9 Grid A (spacing 3.5 m) pre-CPT results. **a** Pre-CPT-13 test results. **b** Liquefaction analysis for pre-CPT-13

versus time plot, reference number of the vibro-compaction probe, date of installation, time required for installation, and maximum depth.

After the vibro-compaction works were completed, the resulting ground-level post-treatment across the area were recorded. The frequency was 1 measurement per 900 m^2 (using a 30 m by 30 m grid). In the case of vibro-compaction, the cone-shaped depression around the vibrators and its extension tubes were continuously filled with the granular material from the existing ground surface using a shovel loader. The post-compaction ground was thus lower than the pre-treatment working platform. Alternatively, the depression can be filled with suitable sand material with a maximum fine content of 10% from a stockpile located on site provided for this purpose.

Post-compaction CPT

Elevations of the ground before and after compaction were documented within every grid. After the vibro-compaction process was finished, ground measurements were collected to assess soil subsidence. The recorded ground levels are shown in Table 3.

Raju and Sondermann (2005) reported that during deep vibro-compaction works, the anticipated settlement after compaction is generally about 5–10% of the compaction depth. In this specific case, the value was about 7.5% and in accordance with the suggested range. Nineteen post-compaction CPT tests in each trial grid were conducted. To evaluate the results in detail, the cone resistance and friction ratio are plotted separately for each test along with

Table 3 Post-compaction ground levels (HSSG-Keller FTR 2010)

Triangular grid (m)	Average depth (m)	Pre-existing ground level	Post-existing ground level	Settlement (m)	% of depth
3.50	7.030	1.463	0.913	0.550	7.80
4.00	6.86	1.492	0.895	0.507	7.39
4.50	6.90	1.502	0.983	0.519	7.52

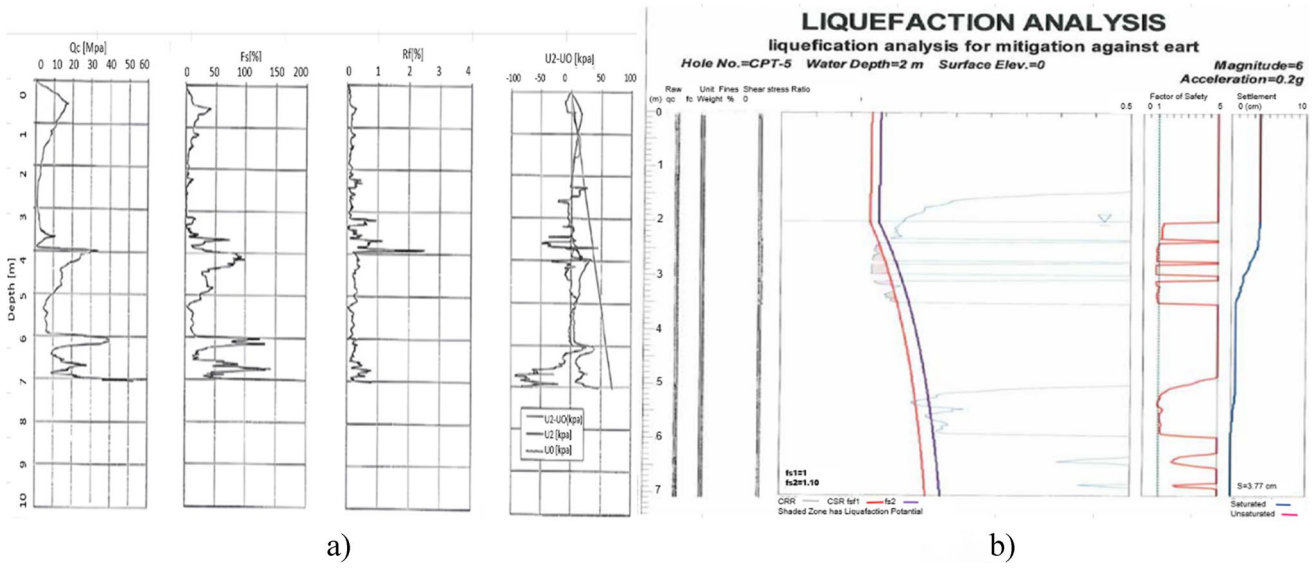


Fig. 10 Grid B (space 4.0 m) pre-CPT results. **a** Pre-CPT-5 test results **b** Liquefaction analysis for pre-CPT-5

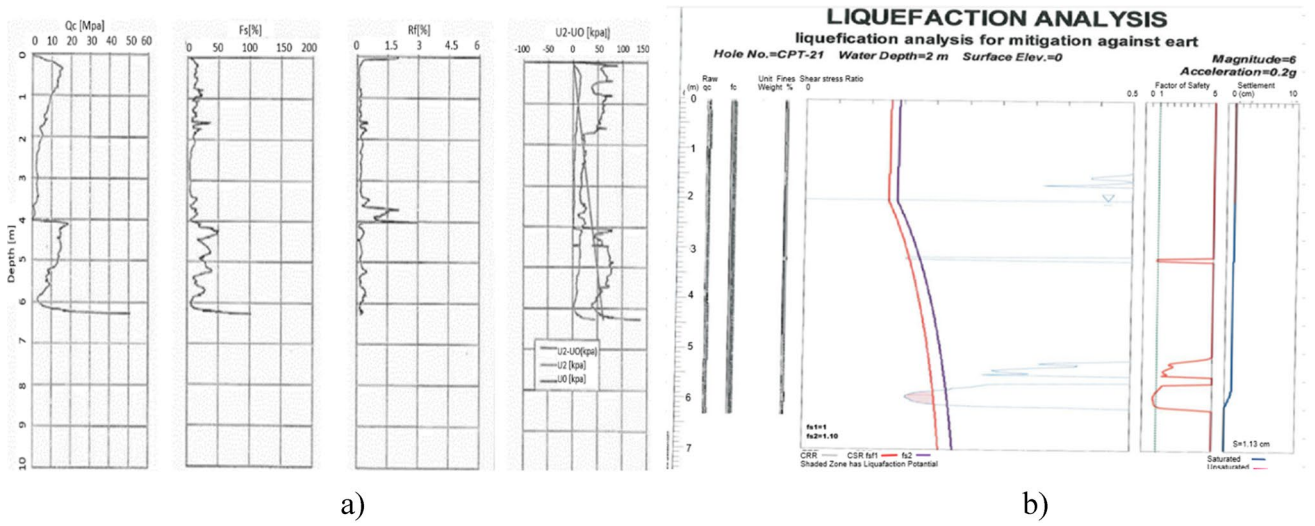


Fig. 11 Grid C (space 4.5 m) pre-CPT results. **a** Pre-CPT-21 test results. **b** Liquefaction analysis for pre-CPT-21

target qc line and 0.5% friction ratio line which indicates the higher fine content. All post-CPT points are illustrated in Fig. 16.

After the vibro-comaction works were completed in the trial areas, post-CPT tests were conducted at specified intervals to monitor the effectiveness of soil improvement. The post-CPT-16 test was performed near the area Grid B and pre-CPT-13 point, and the post-CPT graphs and analyses are shown in Fig. 17. The result graphs of post-CPT-22, which was conducted near pre-CPT-5, and post-CPT-48, which was carried out near pre-CPT-21, are presented in Figs. 18 and 19, respectively.

Results and discussion

The liquefaction analysis was carried out using the data from pre- and post-CPT tests. The liquefaction analyses were based on the UBC-97 category 2A with PGA of 0.20 g. The CPT results were assessed according to the criteria set by the specifications of BS 1377-9 (1990) Test 3.1. The pre-CPT was carried out at the 33 points (numbered from 1 to 33 in grid areas A, spacing 3.5 m), B (spacing 4.0 m), and C (spacing 4.5 m). The pre-CPTs showed cone resistance (qc) values of 10–20 MPa at elevation + 1.00 m



Fig. 12 Keller vibro-compaction twin probe S-700 (HSSG-Keller MS 2010)

and qc values of less than 2.70 MPa between elevations 0.00 and—2.50 m with very low qc values with high friction ratio concentrated at elevation—2.50 m.

The high qc values at the surface were from the traffic or as a result of the surface compaction. The plots indicated sandy soils down to elevation from – 4 to – 6 m with a thin layer of cohesive materials concentrated at the elevation of – 2 to – 2.50 m. Robertson (1990) suggested that soil layers with low qc and high friction ratios should be classified from sensitive to fine grained, organic, clay to silty clay based on the qc/Rf ratio. According to guidelines for soils suitable for the vibro-compaction technique (Fig. 6), this layer falls outside the suitable soil for vibro-compaction parameters and showed no improvement. The thickness of the cohesive layer was very thick in the 3.50 m grid trial area when compared with other grids. Some of the liquefaction analyses indicated that liquefaction would occur between the depths of 2 and 4 m and very few showed a thin layer that was susceptible to liquefaction at depth of 6.0 m.

The post-CPT was carried out at the 19 points (numbered from 1 to 19 in grid areas A, spacing 3.5 m). The post-CPTs showed the average CPT results were well above the performance criteria. The individual CPT plots confirm the qc values for the sandy soil were well above the performance

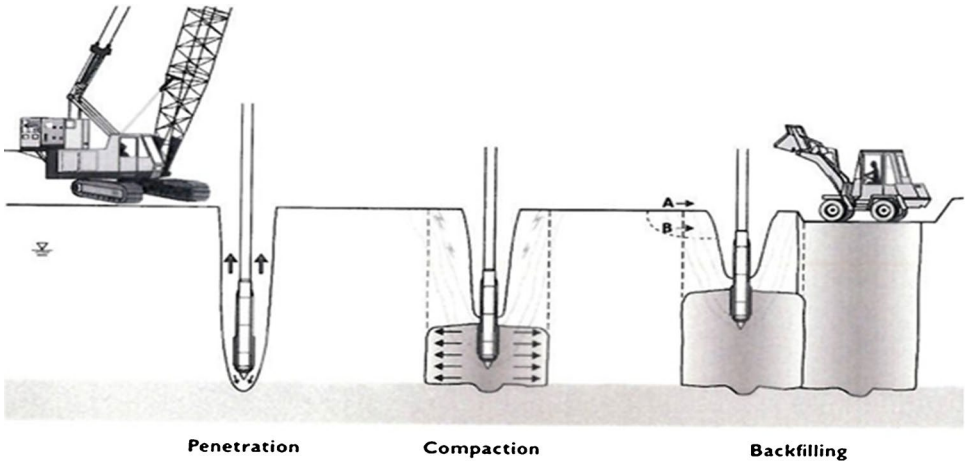


Fig. 13 Vibro-compaction process (HSSG-Keller MS 2010)

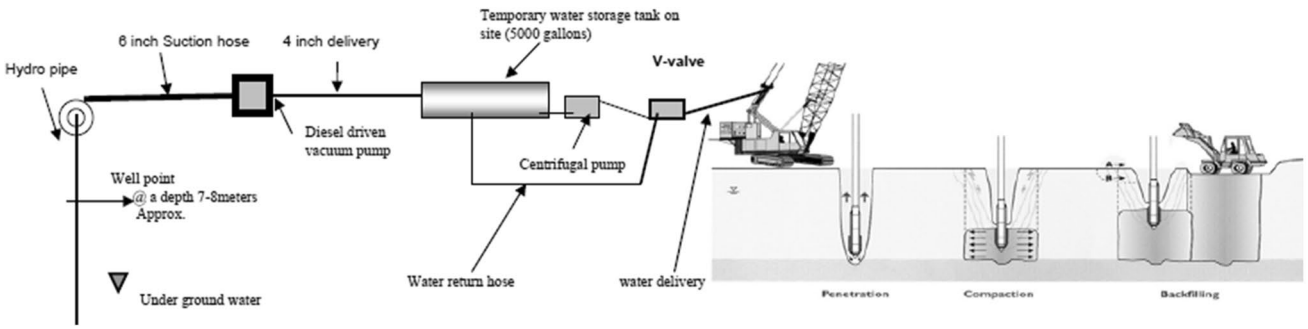


Fig. 14 Vibro-compaction water supply system setup (HSSG-Keller MS 2010)

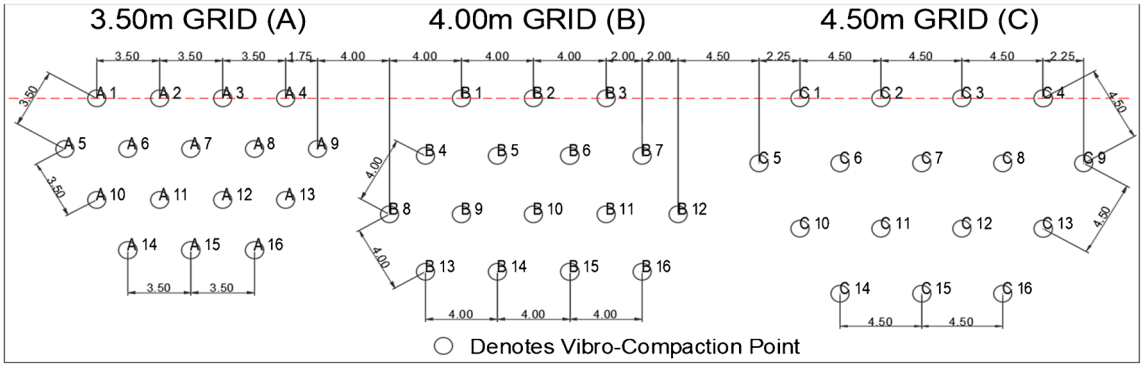


Fig. 15 Trail area vibro-compaction points with grid spacing of 3.50 m, 4.00 m, and 4.50 m

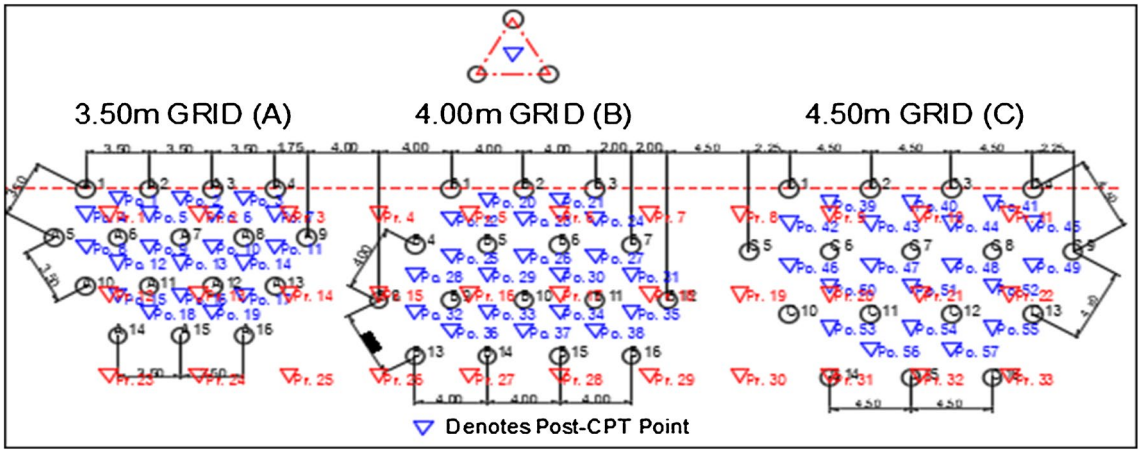
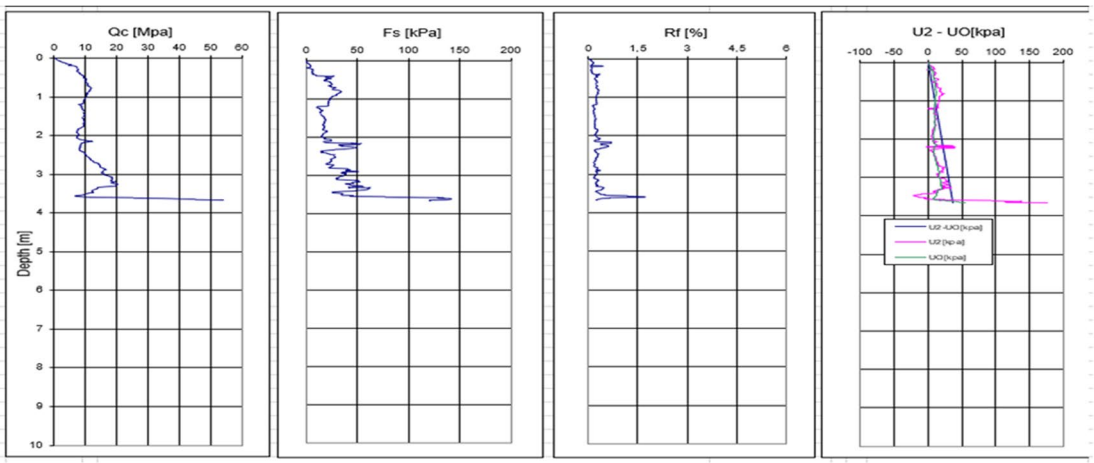


Fig. 16 Set-up scheme of all post-CPT points

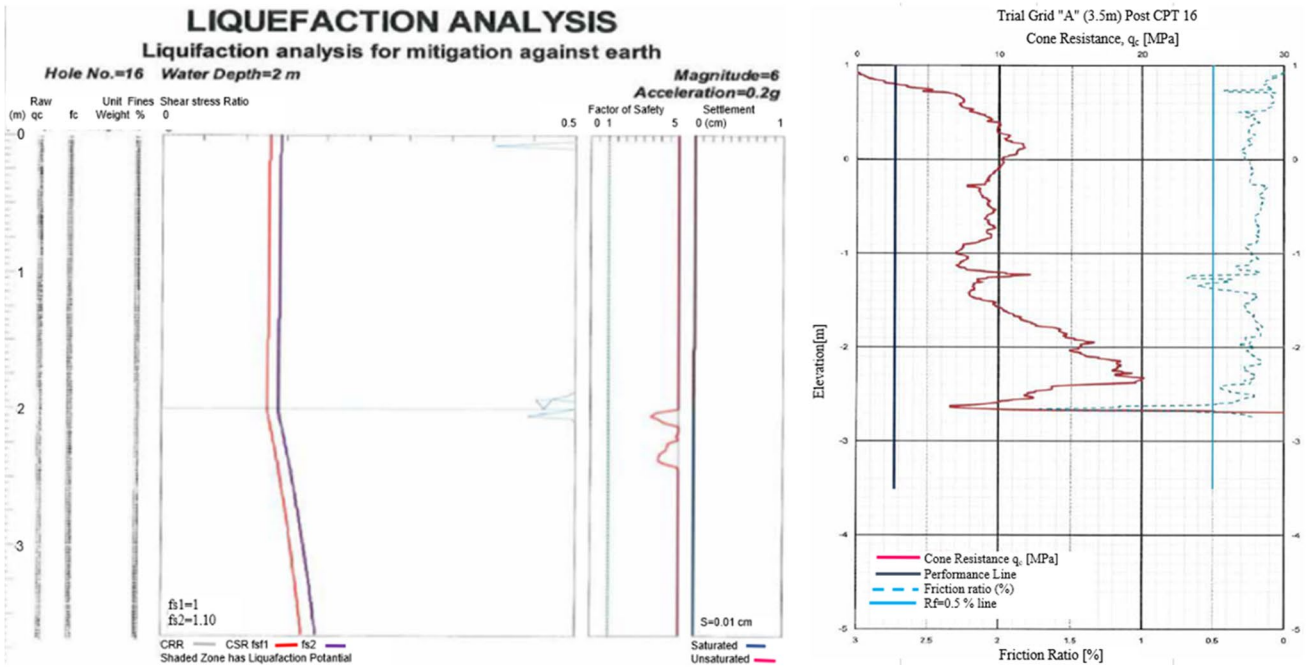
criteria. However, the layers identified in both pre- and post-CPT between – 1.00 and – 3.00 m show the presence of non-compactable materials indicated by high friction ratio as high as 3%. Based on the post-CPT test results, the effect in all the sandy soils considerably exceeded the performance criteria, confirming good vibro-compaction. There was no marginal improvement in the incompatible (non-compactable) soil layers. The thickness of the non-compactable soil layer was thicker in a few tests when compared with Grid B. The liquefaction analysis from 1 to 19 confirmed the liquefaction potential was mitigated successfully.

In Grid B (4.0 m), post-CPT tests were carried out at the 19 points numbered from 20 to 38. The post-CPT tests showed the average CPT results were well above the performance criteria. The individual CPT plots confirmed the qc values for the sandy soil were well above the performance criteria. However, once again the thin layer in both the pre- and post-CPT, between – 2.00 and – 3.00 m, showed the presence of non-compactable soils (silt, clay, and sensitive fine-grained material) identified by a high friction ratio. Based on post-CPT test results, all the sandy soils again considerably exceeded the performance criteria, confirming good vibro-compaction. There was no or only marginal improvement in the non-compactable soil layers. The thickness of the non-compactable soil layer was thin and occurred consistently at the same elevation when compared with the 3.50 m grid. The liquefaction analysis encompassing points 20 to 38 confirmed the liquefaction potential was successfully mitigated.

In Grid C (4.5 m), the post-CPT was carried out at the 19 points numbered from 39 to 57. The post-CPT tests showed the average CPT results were above the performance criteria. Half of the individual CPT plots confirmed the qc values for the sandy soil were well above the performance criteria. In the rest of the tests, the cone resistance fell below the target line by a small margin between – 0.50 and – 2.00 m, despite the sandy soil, indicated by a friction ratio of less than 0.50%. However, the thin layer in both pre- and post-CPT between – 2.00 and – 3.00 m showed the presence of non-compactable soils (silt, clay, and sensitive fine-grained material) identified by a high friction ratio. Based on post-CPT results, sandy soils from + 1.00 to – 0.50 m considerably



a)

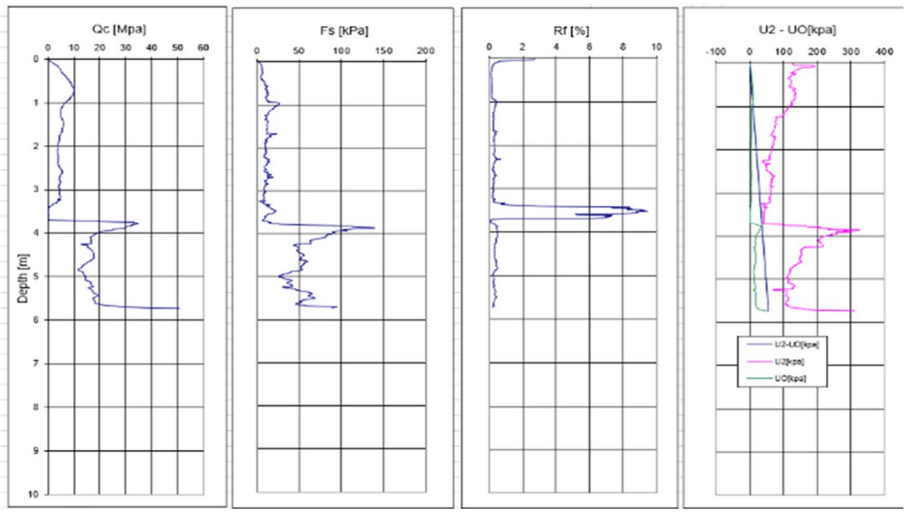


b) c)

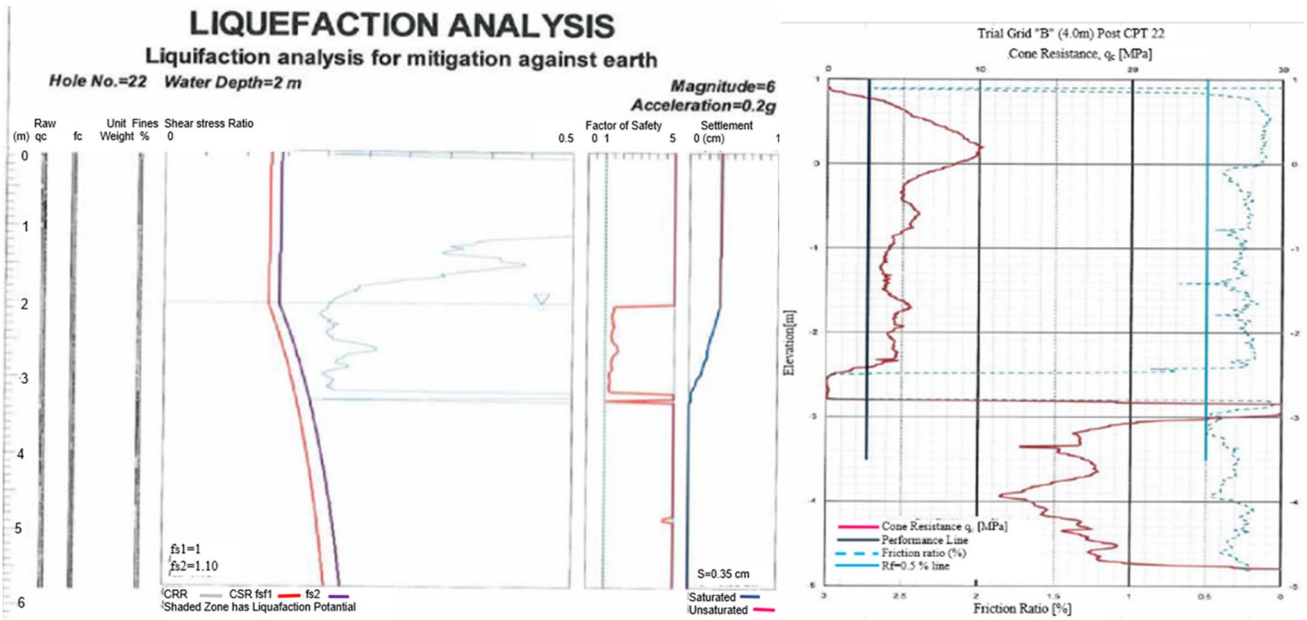
Fig. 17 Grid A (Space 3.5 m) post-CPT results **a**) post-CPT-16 test results **b**) Liquefaction analysis utilizing post-CPT-16 results **c**) Evaluation of cone resistance from post-CPT-16

exceeded the performance criteria. Below this layer, the sandy soil had less effective compaction when compared with the rest of the grids. There was no marginal improvement in the non-compactable soil layers. The thickness of the incompatible soil layer was thicker in a few tests when compared with Grid B. Most of the liquefaction analysis showed the liquefaction potential was mitigated successfully. A few analyses indicated that a thin layer of soils at depths of 3 m and 6 m had a potential to liquefy.

The pre-CPT and post-CPT values and liquefaction analyses of the two closest points within the Grid A vibro-compaction area were compared. These two points are pre-CPT-13 and CPT-16. The results of pre-CPT-13 can be seen in Fig. 9, and the results of post-CPT-16 are shown in Fig. 17. Comparison showed that in pre-CPT-13, the qc value between 2 and 4 m was relatively low, reaching up to 1 MPa. In contrast, in post-CPT-16, within the same range, the qc value increased to an average of 6–8 MPa.



a)



b) c)

Fig. 18 Grid B (Space 4.00 m) post-CPT results a) post-CPT-22 test results b) Liquefaction analysis utilizing post-CPT-22 results c) Evaluation of cone resistance from post-CPT-22

A similar comparison of the pre-CPT and post-CPT values and liquefaction analyses of the two closest points in Grid B, pre-CPT-5 and post-CPT-22, was performed. Figure 10 shows pre-CPT-5 results, and Fig. 18 displays post-CPT-22 results. In the pre-CPT-5, the qc value between 2.5 and 3.5 m was again approximately 1 MPa. However, in post-CPT-22, within the same range, the qc value increases by about 3 MPa. Finally, a similar comparison was performed for the two closest points in Grid C, pre-CPT-21 and post-CPT-48. Figure 11 shows pre-CPT-21 results, and Fig. 19 displays post-CPT-48 results. In pre-CPT-21, the qc value between 3.0 and 4.05 m was approximately 2 MPa. However, in post-CPT-48, within the same range, the qc value increased by about 3 MPa. However, it was notable that the degree of soil improvement in this test area remained relatively low, compared to the test areas with spacings of 3.5 m and 4.0 m.

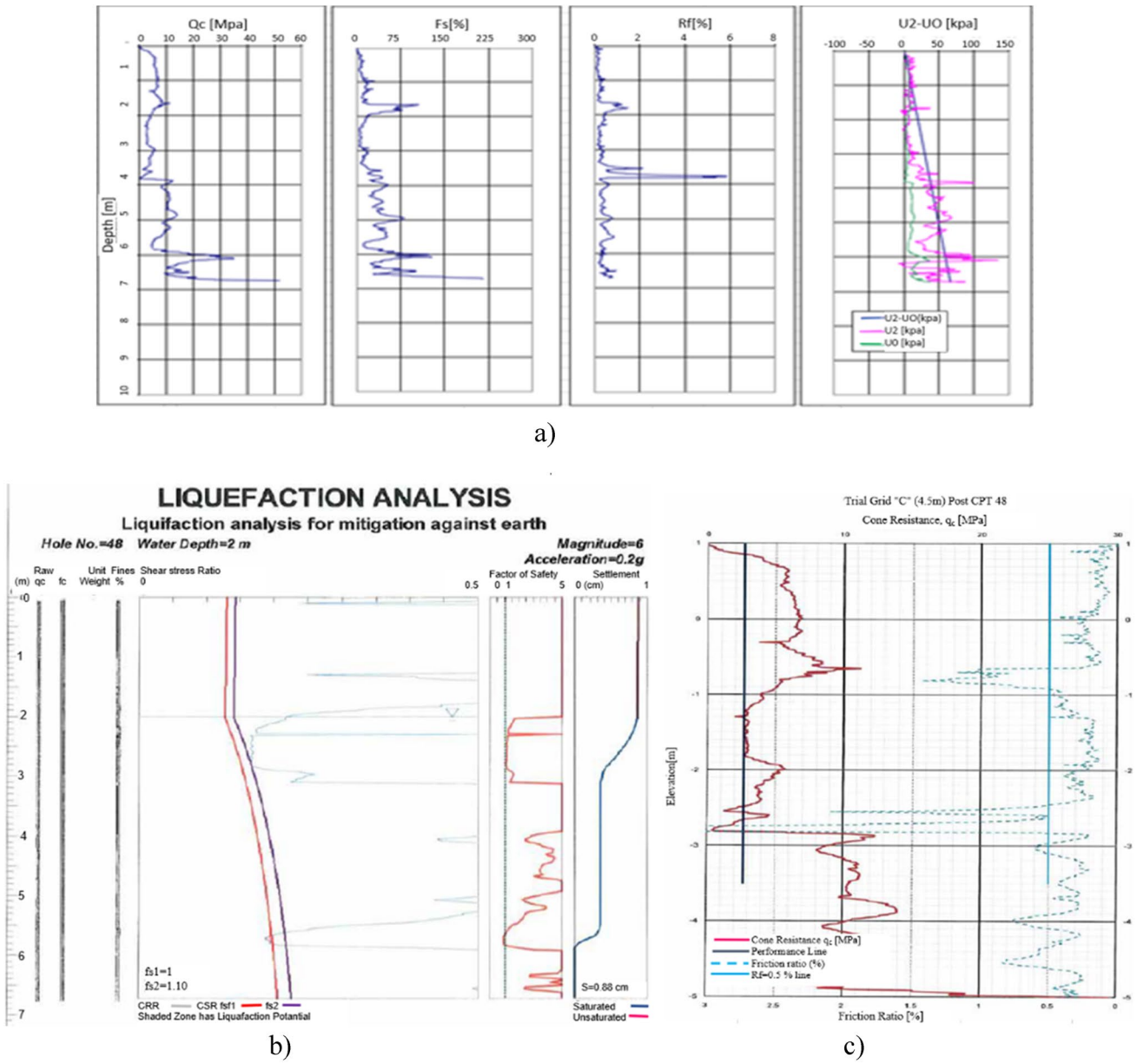


Fig. 19 Grid C (space 4.5 m) post-CPT results. **a** Post-CPT-48 test results. **b** Liquefaction analysis utilizing post-CPT-48 results. **c** Evaluation of cone resistance from post-CPT-48

Conclusions

This study presented a vibro-compaction trial at a site located in Abu Dhabi for soil improvement purposes against the soil liquefaction risk. The CPT soil liquefaction assessment in the field indicated soil liquefaction potential in sandy layers between + 1.5 and − 3.5 m in the study area. The areas with soil liquefaction potential were improved by the vibro-compaction method for increasing the density and bearing capacity of these sandy soils. The technical success of vibro-compaction work was measured by the qc value achieved against a specific cone resistance value target derived from the results of cone penetration tests. Comparisons between pre- and post-compaction tests were conducted, and consistent application parameters were maintained in each case. The test, conducted with three sets of pre- and post-compaction tests using cone penetration test equipment, was carried out across the application points. This trial was applied in three different grid geometries. Grid A (3.50 m) and Grid B (4.0 m) spacings resulted in soil improvement by vibro-compaction, successfully achieving the target results. The effect of vibro-compaction was marginally lower in Grid C with spacing of 4.5 m. Based on the above results, triangular grid B with a spacing of 4.0 m was used for vibro-compaction works. It should be noted that only the sandy layers satisfied the design curve, while

the silty and clay layers fell below the design curve, but without any risk of soil liquefaction. Vibro-compaction was performed at 4125 points across the entire construction site, spaced on a 4×4 m grid, based on trial studies and evaluation of post-CPT results. The effectiveness of the soil improvement works was continuously checked during the field progress by conducting post-CPT tests. The acceptance criteria identified during the site trial stage, along with the results of post-CPTs conducted during the application stage, successfully fulfilled the requirements for mitigating liquefaction potential across the entire project area. As a measurement of the pre- and post-survey, an average settlement of 0.525 m was observed in three trial areas after the vibro-compaction application. The use of vibro-compaction, with the aim of eliminating liquefaction potential by increasing the density of granular fill against dynamic (seismic) effects, also provided additional contributions to the soil. This included enhancing the deformation modulus, reducing total and differential settlement, increasing shear strength to enhance bearing capacity, and contributing to the reduction of permeability.

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Data availability Data is not available for third parties due to restrictions by the HSSG Foundation Contracting LLC.

Declarations

Competing interests The authors declare no competing interests.

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