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Liquefaction induced by deep vertical vibratory compaction

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Three case histories were analysed where three different methods of deep vertical vibratory compaction were used to treat loose, water-saturated sand. Vertically oscillating probes not only generate cylindrical waves with strong ground vibrations that cause compaction of the soil but may also result in liquefaction. Cone penetration tests (CPTs) were performed before and after treatment to evaluate the compaction effect. Soil liquefaction could be observed on all sites during the start of deep vibratory compaction. On two sites, the onset of liquefaction could be documented by vibration measurements. A conventional CPT-based liquefaction analysis was performed, which suggested that liquefaction could be expected at all sites. The case histories analysed in this study suggest that vertically oscillating probes have the potential of being used as full-scale liquefaction testing machines. A new concept is outlined that could be used to establish the liquefaction hazard at a particular site.

Notation

a_{\max}	peak acceleration
C_m	magnitude correction factor
C_q	effective overburden stress adjustment factor
C_u	uniformity coefficient
d_{10}	effective particle size diameter (10% passing)
f	frequency (Hz)
f_s	sleeve resistance
g	acceleration due to gravity
M	earthquake magnitude
q_c	cone resistance
q_{c1}	stress-adjusted cone resistance
R_f	friction ratio
r_d	depth reduction factor
u	pore water pressure
σ_{v0}	vertical total overburden stress
σ'_{v0}	vertical effective stress
σ_{ref}	reference stress (100 kPa)

1. Introduction

In many vibratory compaction projects, the objective is to reduce total and differential settlement by increasing soil density and stiffness. However, an increasing number of applications are related to reducing the risk of liquefaction of loose, water-saturated soils affected by cyclic loading (earthquakes,

wave action, machine foundations or ship impact). Liquefaction susceptibility is usually assessed on an empirical basis, comparing results of penetration tests from sites that have been exposed to earthquake shaking. Seed and Idriss (1971) based liquefaction assessment on the results of standard penetration tests. This concept was later expanded to other in situ testing methods, such as the cone penetration test (CPT), as described by Stark and Olson (1995). Recently, an in situ shaking test (dynamic vibroseis ‘T-Rex’) has been used to examine liquefaction elicited within 3–4 m depth below the ground surface (Stokoe *et al.*, 2014).

In this paper, an alternative concept of in situ liquefaction testing is described where a vertically oscillating probe is used to cause liquefaction along a 10 m long compaction probe. The procedure of vibratory compaction using vertically oscillating compaction probes is first presented. Thereafter, three case histories are described where liquefaction has been observed during treatment. On two of the sites, vibration measurements were taken and used to observe the occurrence of liquefaction. Another case history outlining the concept has been reported by Massarsch and Fellenius (2017a, 2017b). The case histories are analysed with regard to applying vertical compaction probes as testing devices for quantifying liquefaction susceptibility.

2. Deep vertical vibratory compaction (DVVC) methods

Different types of vibratory compaction systems have been developed and are described in the geotechnical literature (Mitchell, 1981; Van Impe *et al.*, 1997). The execution of deep vibratory compaction methods has been standardised in Europe (BSI, 2005). This standard is applicable for the planning, execution, testing and monitoring of ground treatment by deep vibration achieved by depth vibrators and compaction probes. The following treatment methods are covered by this standard: (a) methods in which depth vibrators, containing oscillating weights which produce horizontal vibrations, are inserted into the ground and (b) methods in which compaction probes are inserted into the ground using a vibrator that remains at the ground surface and, in most cases, oscillates in a vertical mode. The most commonly used vibratory compaction method is vibroflotation, which has been extensively described in the geotechnical literature (Bell, 2015). Vibroflotation employs a horizontally vibrating probe (vibroflot) that is first inserted to full depth into the ground and then withdrawn in steps, leaving a densified soil cylinder behind.

DVVC is performed by a heavy, vertically vibrating, construction vibrator attached to the top of a probe. After insertion, the compaction process is started by withdrawing and inserting the compaction probe in repeated steps. During compaction, the soil deposit adjacent to the probe is subjected to strong vertical – but also horizontal – ground vibrations. Different types of DVVC methods have been used in the following case histories described. The first practical application of this method for deep soil compaction, frequently called the ‘Foster’ method, was described by Anderson (1974). During the following four decades, vibratory compaction systems evolved, benefiting from the gradual increase of the availability of powerful and sophisticated vibrators with variable frequency and eccentric moment. Another important step was the development of purpose-designed compaction probes, aiming to optimise the transfer vibration energy from the vibrator to the surrounding soil. Finally, a significant step was the availability of electronic measurement and performance control systems, which can be used to optimise and document the entire compaction process. The evolution and practical application of vertically oscillating compaction probes has been described by Wallays (1982), Massarsch (1991), Massarsch and Westerberg (1995a) and Massarsch and Fellenius (2005). The currently most advanced DVVC method uses the vibration amplification effect that occurs when the vibrator is operated at the resonance frequency of the vibrator–probe–soil system, called ‘resonance compaction’. The application of resonance compaction has been described by Massarsch and Heppel (1991). Resonance compaction has been applied on a variety of projects – for example, the mitigation of liquefaction hazard (Neely and Leroy, 1991), compaction of fill behind a retaining wall (Van Impe *et al.*, 1994), soil improvement for dynamically loaded foundations (Massarsch and Westerberg, 1995b),

compaction of landfill for airport runways (Choa *et al.*, 2001), compaction of landfill on natural soil deposit (Massarsch and Fellenius, 2005), liquefaction mitigation for infrastructure (Liu and Cheng, 2012), increase of horizontal pile resistance (Li *et al.*, 2018) and underwater compaction of sand fill within steel caissons (Massarsch *et al.*, 2017a, 2017b).

A potentially important application of DVVC, which has not been previously recognised, is the possibility of causing the liquefaction of natural or man-made soil deposits under controlled conditions and, therefore, to determine whether or not a sand layer is susceptible to liquefaction. Ground vibrations can be measured at or below the ground surface and provide valuable information about the shear strain level and number of vibration cycles at which liquefaction occurs. DVVC has the potential of being used as a full-scale liquefaction testing method (Massarsch and Fellenius, 2017a, 2017b). Thus, in addition to the visual observation of liquefaction, which can occur during DVVC, vibration measurements on or below the ground surface provide quantitative information regarding liquefaction (Massarsch and Fellenius, 2017a, 2017b). In the following, the results of measurements from three case histories are presented, where different types of DVVC methods were used. Also shown is that vibration measurements during compaction can be used to estimate the shear strain level during vibratory excitation and how liquefaction risk can be evaluated using the results of CPT investigations.

3. VibroWing compaction, Rostock, Germany

The extension of a harbour at Rostock, Germany, required the compaction of 1.2 million m³ of hydraulic sand fill to establish a storage area designed to carry a uniformly distributed load of 300 kPa. The project, which was carried out between 1980 and 1982, also included the construction of a 1250 m long wall consisting of precast concrete caissons with a 14.1 m high wall with a diameter of 15 m. The sand fill had to be compacted to a depth of between 7 and 15 m. A special challenge of the project was the compaction of loose, water-saturated sand fill inside the cylindrical concrete caissons. Details of the project and the execution of ground treatment have been reported by Massarsch and Broms (1983) and Broms and Hansson (1984). The hydraulic fill was reclaimed from the Baltic Sea and consisted of uniform medium sand containing layers of fine sand and coarse sand. The groundwater level before compaction was 1.5 m below the surface of the fill. The sand was generally poorly graded ($C_u = 2.5$) with an effective grain size, d_{10} , of about 0.2 mm. CPTs were performed before compaction of the hydraulic fill and compaction of the fill inside the concrete caissons (Figure 1). Prior to improvement, the cone resistance in the landfill area ranged between 3 and 8 MPa, while, inside the caissons, it ranged between 1.5 and 3 MPa. That is, the cone resistance in the hydraulic fill inside the caissons was about half that in the landfill. After compaction, the cone resistance increased significantly throughout the

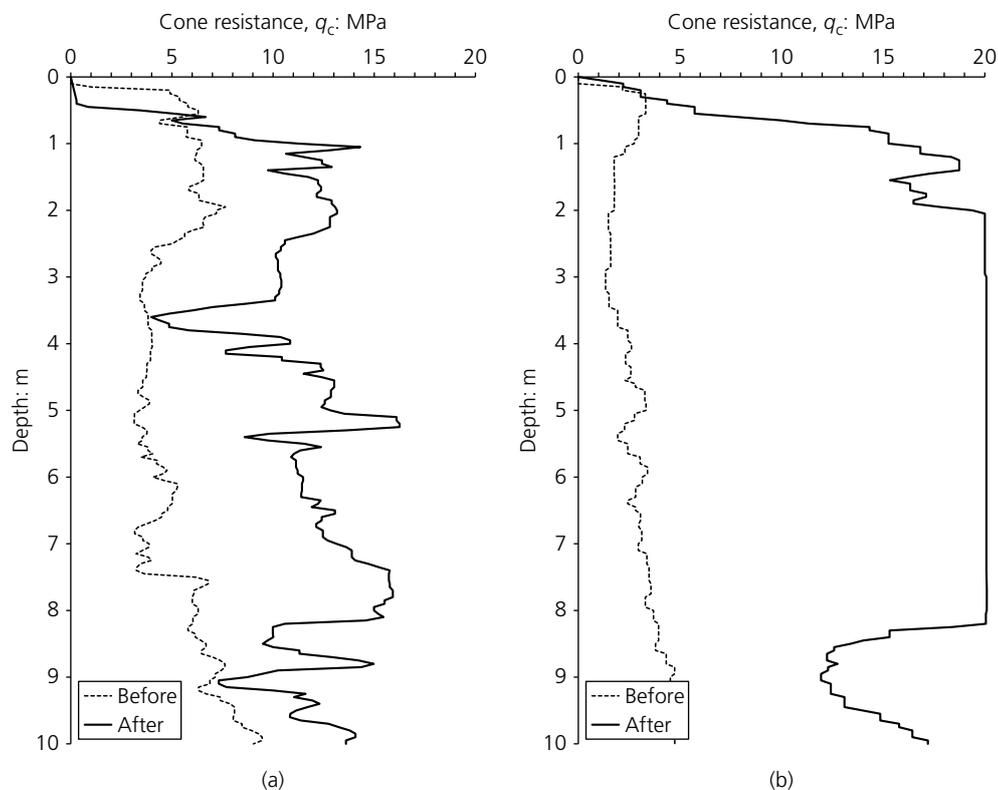


Figure 1. Cone resistance before and after vibratory compaction. Note: the maximum capacity of the CPT was 20 MPa. (a) Tests in the landfill. (b) Tests inside the concrete caisson

soil deposit. Moreover, the increase was more pronounced inside the concrete caissons.

3.1 Compaction method

The loose hydraulic fill was compacted between 7 and 15 m depth by the VibroWing method (described by Massarsch and Broms, 1983). The VibroWing equipment consisted of a 15 m long steel rod, provided with 0.85 m long wings of steel spaced about 0.5 m apart (Figure 2). An electric vibrator with variable frequency (Tomen VM2-5000A) was attached to the top of the compaction probe and suspended from a crane with a lead. The performance characteristics of the vibrator are given in Table 1.

During compaction trials, the operating frequency of the vibrator was varied and it was found that the most efficient compaction was achieved at a frequency of 20 Hz. Extensive field measurements were performed, including ground vibration measurements, cross-hole tests and settlement measurements (Massarsch and Lindberg, 1984).

3.2 Monitoring of compaction

Compaction was carried out in two passes in a triangular pattern at a spacing of 2.5 m, with compaction points during the secondary pass at the centre of the primary grid. The

duration of compaction after full probe insertion was approximately 5 min. As a result of compaction, the ground surface settled by about 0.5 m. During the initial treatment of the loose sand, it was noted that the water level in the settlement trough temporarily rose to a level more than 1 m above sea level (Figure 2). Different types of seismic measurements were performed to study the compaction process. Figure 3 shows the vertical and horizontal vibration velocities, measured by geophones at selected distances from the compaction point. Vibrations at 2 m distance were measured only in the vertical direction. Ground vibrations varied during the compaction process and decreased with increasing distance. The highest vibration velocity (27 mm/s in the vertical direction) was measured at the ground surface at 2 m distance. It is interesting to note that vibrations were higher in the horizontal direction despite the use of a vertically oscillating probe. It should be pointed out that, at the time of the project (1981), vibration measurement on construction sites was still a challenging task and vibration measurements could only be performed intermittently.

3.3 Manifestation of liquefaction

During the initial compaction phase (pass 1), the ground surface subsided and the groundwater level rose by approximately 1.5 m, as can be observed in Figure 2(b).



(a)



(b)

Figure 2. Compaction with VibroWing in hydraulic fill at harbour project, Rostock, Germany (source: Massarsch and Lindberg, 1984): (a) VibroWing compaction probe; (b) subsidence and liquefaction due to compaction

In compacting the very loose, water-saturated sand fill inside the concrete caissons, the water level was about 2 m below the upper rim of the caissons. The same compaction equipment and treatment method were used inside and outside the caissons. Compaction inside the caissons started with the probe inserted in the centre. During initial compaction, with the probe having penetrated approximately 10 m depth, the sand fill inside the concrete cylinder liquefied spontaneously, with

Table 1. Performance characteristics of electric vibrator with variable frequency (Tomen VM2-5000A)

Property	Value	Unit
Eccentric moment	50	kg·m
Max. centrifugal force	700	kN
Max. frequency	20	Hz
Max. amplitude (peak to peak)	18	mm
Max. static line pull	300	kN
Dynamic mass	5500	kg
Total mass (without clamp)	7000	kg

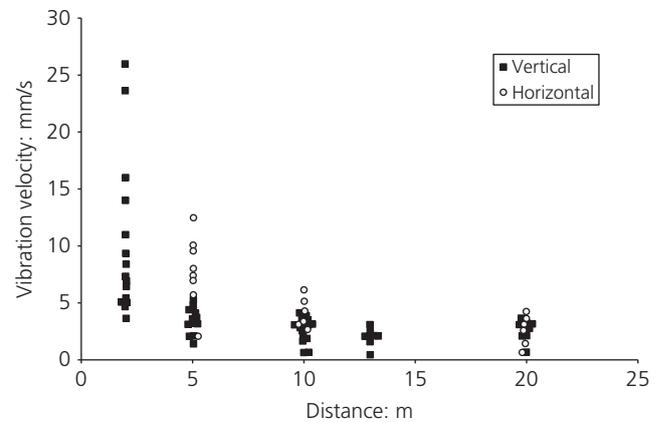


Figure 3. Vertical and horizontal vibration velocity with increasing distance from compaction point, after Massarsch and Lindberg (1984)

springs ('boils') emerging at the ground surface (Figure 4). During this phase of compaction, ground vibrations ceased completely, indicating complete loss of soil strength due to excess pore water pressure. It is apparent that the concrete caissons restricted drainage in the horizontal direction, allowing pore pressure to build up. During liquefaction, compaction was occasionally interrupted and continued after the dissipation of excess pore water pressure, which occurred within a few minutes.

4. Y-Probe compaction, Broechem, Belgium

The Y-probe system was used for the first time in Broechem near Antwerp, Belgium (Wallays, 1982) (Figure 6). A hydraulic sand fill 10 m deep had to be compacted. The groundwater level was located approximately 2 m below the ground surface. The soil consisted of fine- to medium-grained sand. The grain size distribution was within the lower range of the grain size distribution recommended by Mitchell (1981) as being suitable for compaction. The effective particle diameter d_{10} varied between 0.03 and 0.12 mm. The geotechnical conditions were investigated using a mechanical CPT. The results before and after compaction are shown in Figure 5. Prior to compaction, the cone resistance down to about 4 m depth ranged between $q_c \approx 4$ and 7 MPa. After about 4 m depth, a deposit of loose silty sand and sand with $q_c \approx 2-4$ MPa followed. The design



Figure 4. Water flow as evidence of liquefaction during compaction of the hydraulic fill inside the concrete caisson (source: Massarsch and Lindberg, 1984)

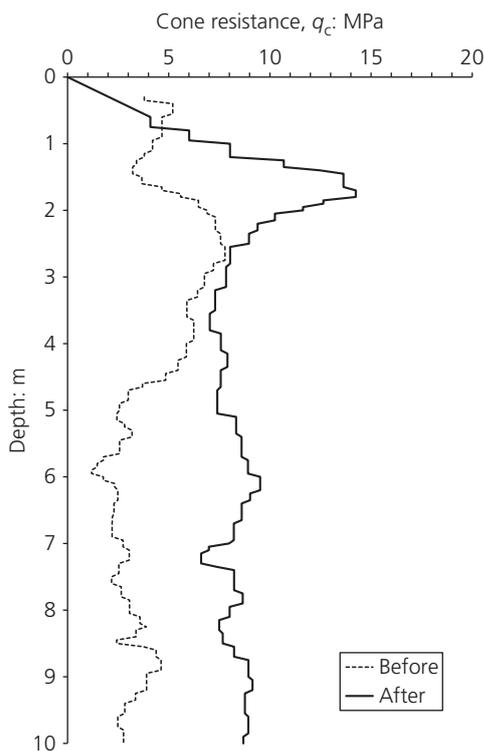


Figure 5. Results of the mechanical cone penetrometer before and after compaction at a 3.5 m triangular grid

requirement was to increase the cone resistance to $q_c > 8$ MPa. After treatment, the cone resistance increased to between 7 and 10 MPa.

4.1 Compaction method

The 15 m long compaction probe was Y-shaped with three 500 mm wide and 2 mm thick steel blades. Horizontal steel ribs were welded to the blades at a vertical distance of 2 m. Compaction was carried out with a PTC 40 A2 vibrator, operated at a constant frequency of 25 Hz. The performance characteristics of the vibrator are given in Table 2.

4.2 Manifestation of liquefaction

Compaction trials were performed in a triangular grid at various spacings of 2.5, 3.5 and 4.5 m. During the initial phase of compaction, the groundwater rose by about 1.5 m and liquefaction was observed in the form of sand boils (Figure 6).



(a)



(b)

Figure 6. Y-probe compaction of hydraulic sand fill consisting of loose silty sand at Broechem, Belgium, from Wallays (1982): (a) Sand boils after vibratory compaction; (b) manifestation of liquefaction during compaction

Table 2. Performance characteristics of vibrator with variable frequency (PTC 40 A2)

Property	Value	Unit
Eccentric moment	40	kg·m
Max. centrifugal force	1000	kN
Frequency	26	Hz
Max. amplitude (peak to peak)	30	mm
Max. static line pull	N/A	kN
Vibrating mass	2600	kg
Total mass (without clamp)	N/A	kg

5. Double-Y probe resonance compaction, Map Ta Phut, Thailand

Ground improvement was required for the development of a tank farm terminal along the sea shore at Map Ta Phut, Thailand. The purpose was to limit the total and differential settlement of the hydraulic fill. The project started in 1995 and resonance compaction was used to improve the soil deposit. Extensive geotechnical and seismic field investigations were carried out to optimise the compaction process (Massarsch and Westerberg, 1995a; Sandberg and Törnbohm, 1996). Hydraulic fill was placed by pumping slurry from different locations around the perimeter of the site. Due to this method of fill placement, the soil deposit had a layered structure, with fine-grained (silty and clayey) material deposited further away from the outlet. The thickness of the reclaimed material was approximately 10 m. The groundwater table was located approximately 4 m below the ground surface. The geotechnical properties of the soil deposit were investigated by

sampling and CPTs. The soil deposit consisted of a 3 m thick crust of fine to dense sand with silt, followed by 6 m of loose fine sand. At 9 m depth, loose clayey sand was encountered, resting on stiff silty clay and clayey sand. Results from CPTs before and after treatment are shown in Figure 7. The test results before compaction showed a surface crust with a cone resistance, q_c , between 10 and 15 MPa. The relatively stiff layer was the result of desiccation. Below the groundwater level at 4 m depth, the cone resistance decreased to values ranging from 3 to 7 MPa. The layered structure of the hydraulic fill was apparent, with layers of silt and clay embedded in the sand fill. The sleeve resistance, f_s , shows a similar layered soil structure to the cone resistance. The friction ratio, R_f , indicated several fine-grained layers (silt and clay) with $R_f > 1.5\%$.

Following compaction, the cone resistance increased from 10 to 30 MPa. The improvement effect was influenced by the occurrence of fine-grained layers, which were apparent from the high values of the friction ratio. It is important to note that the sleeve resistance also increased, in many cases significantly more than the cone resistance. The significance of horizontal stress increase for settlement analyses has been described by Massarsch (1994).

5.1 Compaction method

The ground treatment method chosen by the contractor was resonance compaction, using a double Y-probe, provided with four 0.8 m wide wings, as shown in Figure 8. The length of the compaction probe was 15 m. A special feature of the probe design was the incorporation of circular openings in the wings

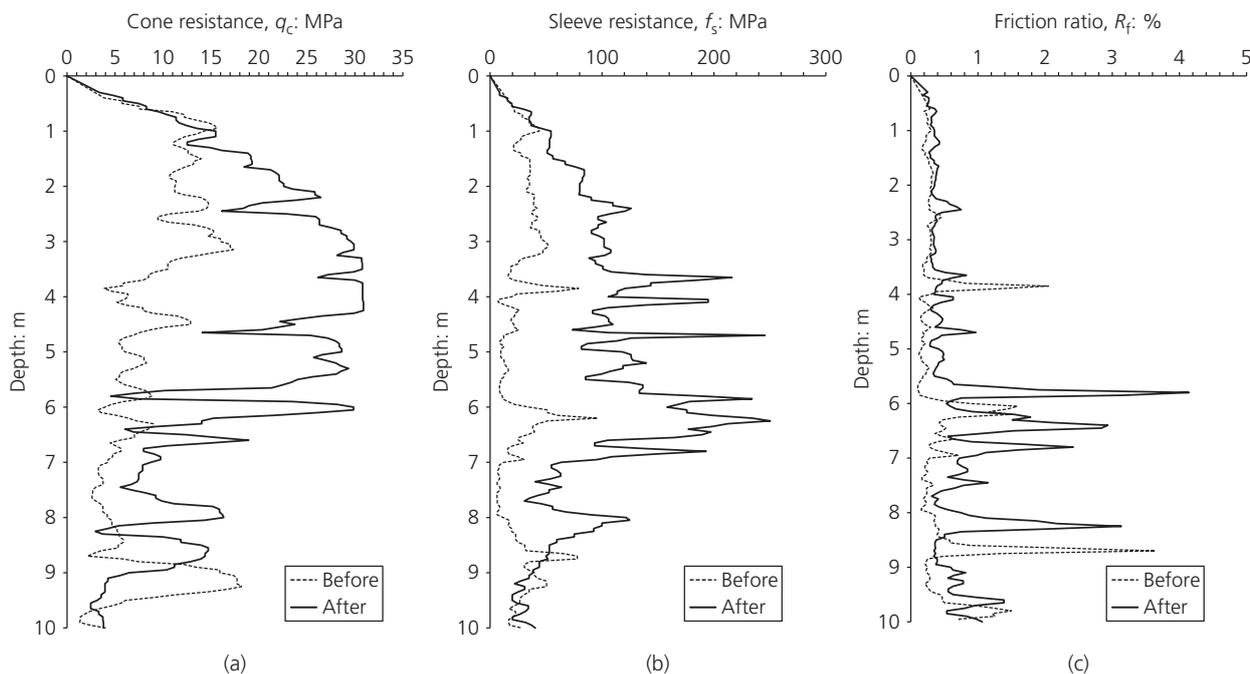


Figure 7. Results of CPTs in the trial area: (a) cone resistance; (b) sleeve resistance; (c) friction ratio



Figure 8. Resonance compaction machine with flexible, double-Y compaction probe at Map Ta Phut, Thailand (source: Massarsch and Westerberg, 1995a)

Table 3. Performance characteristics of hydraulic vibrator with variable frequency and step-wise adjustable eccentric moment (Müller MS100 HF)

Property	Value	Unit
Eccentric moment (in steps)	48/60/80/100	kg-m
Max. centrifugal force	670	kN
Frequency steps	36/32/27-8/25	Hz
Max. amplitude (peak to peak)	26	mm
Max. static line pull	600	kN
Vibrating mass	7700	kg
Total mass (without clamp)	10 900	kg

and the flange. Thereby, the probe weight could be reduced significantly and the interaction between the probe and the soil enhanced. A powerful, hydraulic vibrator with step-wise variable eccentric moment and variable frequency (Müller MS100 HF) was used (Table 3). Thus, it was possible to increase the eccentric moment (by adding mass) when operating the vibrator at a lower (resonance) frequency. The performance characteristics of the vibrator are given in Table 3.

5.2 Monitoring of compaction

At the start of the project, compaction trials were carried out at different compaction point spacings. During the compaction trials, the resonance frequency of the vibrator–probe–soil system

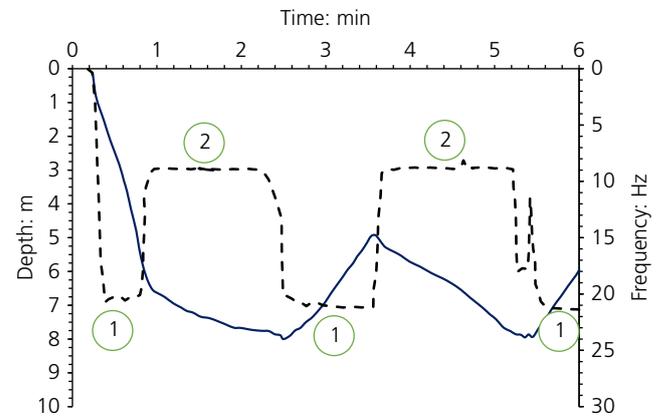


Figure 9. Compaction probe movement (solid line) and vibrator operating frequency (dashed line); probe penetration and extraction at high frequency (20 Hz) and resonance compaction at resonance frequency (10 Hz). Corresponding ground vibration measurements are shown in Figure 10

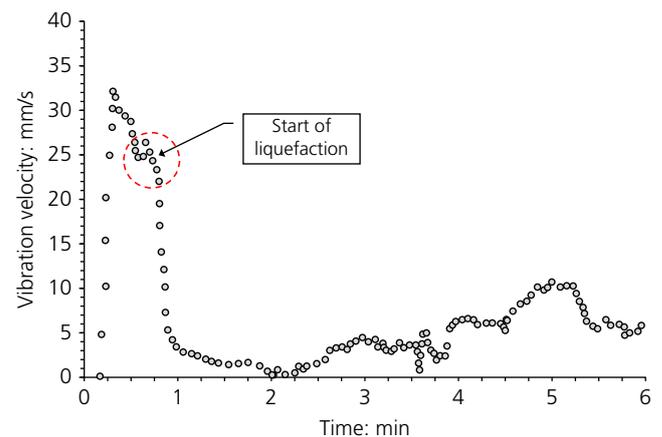


Figure 10. Vertical vibration velocity measured 6 m from compaction point (see Figure 9). The onset of liquefaction is clearly apparent by the sudden drop in ground vibrations

was determined (Massarsch and Westerberg, 1995a, 1995b). Soil resonance occurred at a frequency of 10–15 Hz and was influenced by the size and length of the compaction probe. Typically, the resonance frequency increased during the second compaction pass, indicating an increase in soil stiffness (shear modulus). The resonance compaction process is illustrated in Figure 9. The probe was inserted to 7 m depth ① at high frequency (20 Hz). The vibrator frequency was then lowered to 10 Hz to achieve resonance ②. Thereafter, the probe was extracted at high frequency ① and re-inserted at low frequency ②. Finally, the probe was extracted at high frequency ③.

5.3 Manifestation of liquefaction

The vertical vibration velocity was measured at 6 m distance from the compaction probe, which was compacting the loose sand at about 5 m depth (Figure 10). During probe penetration



(a)



(b)

Figure 11. Liquefaction during the first pass of resonance compaction in the hydraulic fill at Map Ta Phut: (a) groundwater rising along the compaction probe; (b) accumulation of silty sand at the ground surface

through the stiff surface layer, strong ground vibrations were recorded (>30 mm/s). However, when the probe reached the loose sand layer and the vibration frequency was lowered to the resonance frequency (10 Hz), ground vibrations suddenly dropped to very low values, indicating liquefaction. Due to the sudden liquefaction of the loose sand layer, groundwater propagated along the shaft of the compaction probe 4 m into the ground surface (Figure 11(a)). The water contained silt and clay particles. After the first compaction pass, water accumulated on the ground surface, as shown in Figure 11(b). The fine-grained layers in the hydraulic fill most likely contributed to soil liquefaction as these low-permeability layers slowed the vertical dissipation of excess pore water pressure, allowing the pore pressure to increase.

6. Analysis of DVVC case histories

Three case histories have been presented in which loose, water-saturated soil deposits were treated by different DVVC methods: VibroWing, Y-probe and double Y-probe. These three sites have been analysed with respect to the liquefaction

potential, based on generally accepted concepts initially proposed by Seed and Idriss (1971) and expanded to the use of CPTs (Seed and de Alba, 1986). Stark and Olson (1995) presented relationships between the cone penetration resistance and liquefaction potential of sandy soils for liquefaction assessment. CPT-based relationships were developed for clean and silty gravelly sands based on 18 liquefaction case histories and one non-liquefaction case history. The concept of analysis is applied in the following sections to evaluate the liquefaction potential of a 'typical site' treated by DVVC.

6.1 Vibrator performance characteristics

The vibrator performance characteristics of the DVVC methods used in the three sites were presented above. For the purpose of a typical or representative analysis, the following parameters were assumed. The centrifugal force of the vibrators ranged from about 700 to 1000 kN (average 800 kN). The movement amplitude (peak-to-peak) of the suspended vibrator ranged from 20 to 30 mm (average 25 mm). The compaction frequency varied between 15 and 25 Hz (average 20 Hz). If it is assumed that the duration of effective compaction lasted at least 5 min (not including penetration and extraction), the number of compaction cycles in each compaction point would exceed 6000. This number can be compared with the duration of strong shaking during a large earthquake lasting approximately 20 s, resulting in an equivalent number of cycles rarely exceeding 40. The vibration velocity of a suspended vibrator and probe, operating at 20 Hz and with a movement amplitude of 12 mm (single amplitude), can be estimated to be 1570 mm/s ($20g$). During the penetration of a probe, a large part of the vibration energy will be consumed by resistance along the probe shaft. In contrast, in the case of resonance compaction, the probe and soil oscillate 'in phase' and, therefore, in the case of resonance compaction, the loss of vibration energy is minimal.

6.2 Vibration response of the ground during compaction

It is difficult to estimate the transfer of vibration energy from a probe to surrounding soil theoretically. However, the vertical vibration velocity generated by the cylindrical waves emitted from the probe can be measured. Examples of such vibration measurements are shown in Figures 3 and 10. In the case of earthquake loading, the soil is subjected to horizontal cyclic strain. Another difference is that the vibration frequency during DVVC is about ten times higher than that during an earthquake. However, as has been demonstrated by the above case histories, loose water-saturated soils do liquefy in a manner similar to that during earthquake loading. Dobry *et al.* (1982) showed that excess pore water pressure develops when the peak shear strain, γ , is greater than 0.01% (the threshold γ). Exceeding the threshold shear strain results in a restructuring of granular soils (and thus liquefaction), as discussed by Drnevich and Massarsch (1979).

From ground vibration measurements, it is possible to estimate the shear strain, if the shear wave speed of the soil can be measured or estimated. In a soil cylinder of 5 m radius surrounding the compaction probe, the average vibration velocity is typically 20 mm/s. At a frequency of 20 Hz, the corresponding vertical ground acceleration is $0.26g$ (2510 mm/s^2). It can be assumed that, at a distance of 5 m, the vertical vibration velocity generally exceeds 15 mm/s, but is significantly higher closer to the compaction point (Massarsch and Fellenius, 2005). If the average shear wave speed of loose (uncompacted) sand is around 150 m/s (Massarsch and Broms, 1983), the shear strain amplitude is higher than 0.01%. However, closer to the compaction probe, the shear strain level exceeds 0.05% shear strain.

6.3 Correction of cone resistance

Most field observations of liquefaction occur at a vertical effective overburden stress between 50 and 120 kPa. To account for the influence of vertical effective stress, the cone resistance, q_c , needs to be corrected with respect to a reference vertical effective stress of 100 kPa. Stark and Olson (1995) proposed determining the stress-adjusted cone resistance, q_{c1} , from

$$1. \quad q_{c1} = C_q q_c$$

where C_q is the effective overburden stress adjustment factor, which can be calculated from

$$2. \quad C_q = \frac{1.8}{0.8 + (\sigma'_{v0}/\sigma_{ref})}$$

where σ'_{v0} is the vertical effective stress and σ_{ref} is the reference stress (100 kPa). The C_q values are slightly larger at a shallow depth than the C_N value proposed by Seed and Idriss (1971). In Figure 12, the stress-adjusted cone resistance, q_{c1} , determined according to Equations 1 and 2, is shown for the three case study sites. The lowest q_{c1} values were obtained between 6 and 8 m depths. At this depth, the vertical effective stress ranged between about 50 and 80 kPa.

6.4 Estimation of seismic shear stress ratio (SSR)

The SSR in a soil cylinder surrounding a compaction probe can be estimated using the simplified method proposed by Seed and Idriss (1971). Stark and Olson (1995) used the following procedure to calculate the SSR at any point in the ground

$$3. \quad \text{SSR} = 0.65 \frac{a_{\max} \sigma_{v0}}{g \sigma'_{v0}} r_d$$

where a_{\max} is the peak acceleration measured at the ground surface, g is acceleration due to gravity (9.81 m/s^2), σ_{v0} is the vertical total overburden stress, σ'_{v0} is the vertical effective overburden stress and r_d is the depth reduction factor. The depth reduction factor can be estimated in the upper

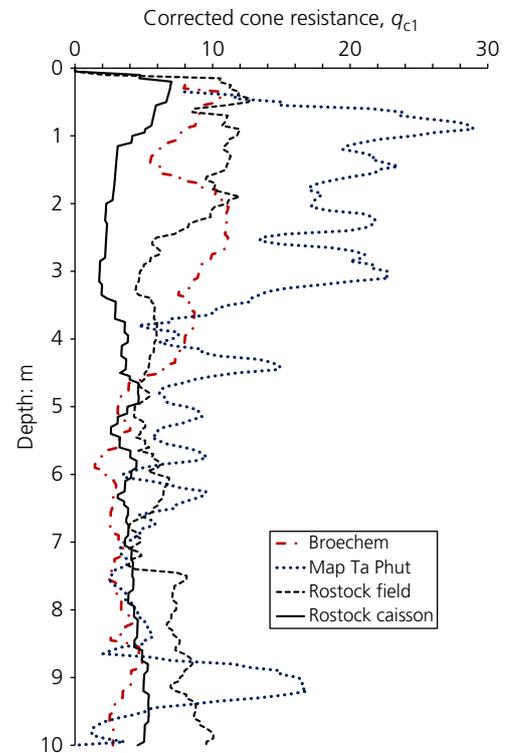


Figure 12. Stress-adjusted cone resistance according to Equations 3 and 4 for project sites

10 m of soil as

$$4. \quad r_d = 1 - (0.0012z)$$

where z is the depth in metres. The SSR was corrected to an earthquake magnitude of 7.5 using a magnitude correction factor, C_m . The SSR was determined for geotechnical conditions representing the three test sites. A vertical effective stress of 70 kPa was assumed, which corresponds approximately to 6 m depth. At 6 m depth, according to Equation 4, the reduction factor is $r_d = 0.93$. Assuming a vertical vibration velocity of 20 mm/s and a vibration frequency of 20 Hz, the vertical ground acceleration adjacent to the compaction probe is $20g$. The ratio of vertical total stress to vertical effective stress, as described in Equation 3, is approximately 1.3 (90/70). Accordingly, the SSR adjacent to a vertically oscillating probe is estimated to be 0.3. Figure 13 shows the CPT-based liquefaction diagram proposed by Stark and Olson (1995). At a depth of 6 m, the SSR was estimated to be 0.3, showing that all three sites would liquefy during vibratory compaction.

7. Recommended liquefaction monitoring concept

The liquefaction monitoring concept outlined above can be readily implemented. It is based on the sensors shown in

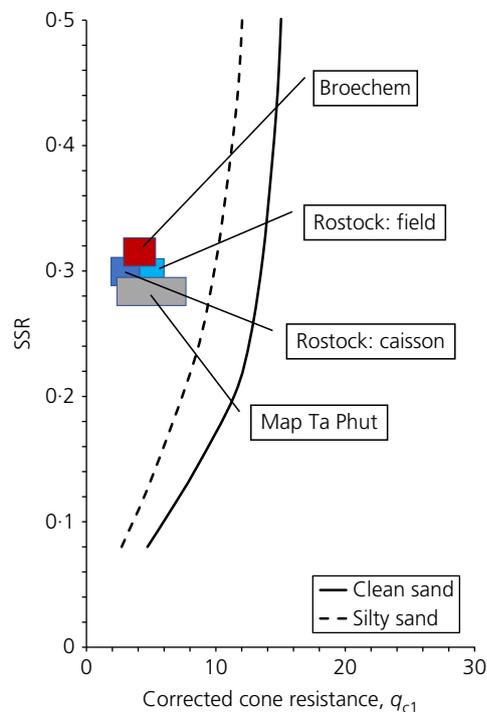


Figure 13. CPT-based liquefaction potential relationship, after Stark and Olson (1995). Boundary of liquefaction for earthquake magnitude $M=7.5$ in clean sand and silty sand. The SSR (0.3) was estimated at 6 m depth for the different sites

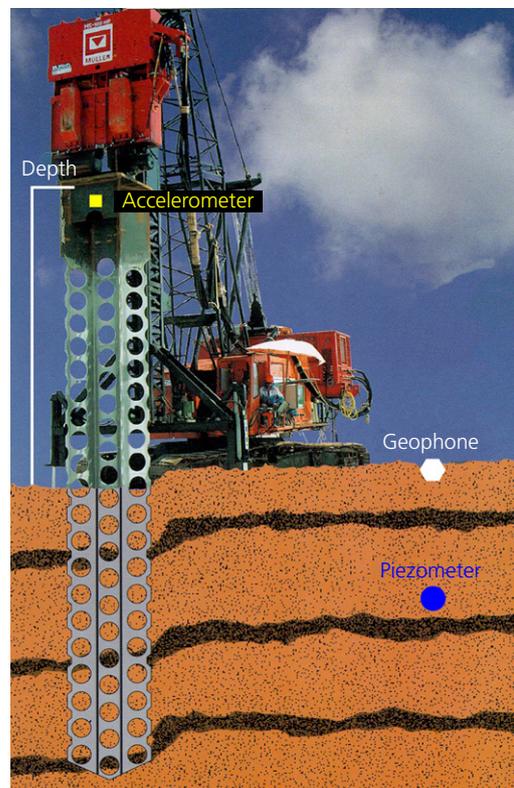


Figure 14. Conceptual illustration of the liquefaction monitoring system

Figure 14 and consists of the following components: (a) a depth-measuring system (records probe depth and penetration speed); (b) an accelerometer mounted on the vibrator (measures acceleration and vibration frequency from which the movement amplitude of the probe can be determined); (c) ground vibrations on the ground surface and/or below the ground surface monitor the ground vibration velocity in three directions (from which the system resonance frequency can be determined); (d) a piezometer at one or several depths below the ground surface to verify the occurrence of liquefaction and pore water dissipation. Based on past experience, it is recommended to perform vibration measurements and pore water pressure measurements at a distance of 4 m from the centre of the compaction point. It is suggested to perform a resonance compaction test prior to the start of the actual compaction as well as after the completion of the treatment.

The cyclic strain method proposed by Dobry *et al.* (1982) is readily applicable for the interpretation of the measurement results. It can be assumed that liquefaction is caused by the vertical component of the cylindrical waves emitted from the probe. From the ground vibration measurements, the vibration frequency and the number of equivalent vibration cycles can be determined.

Down-hole or cross-hole seismic measurements can be readily performed to determine the shear wave speed of the soil prior

to and after compaction. Alternatively, the shear wave speed can be estimated with sufficient accuracy and is typically around 100 m/s for uncompacted sand and increases by approximately a factor of 2 after treatment. Knowing the shear wave speed, the shear strain level can be readily calculated.

8. Conclusions

Vertical vibratory compaction was carried out on three sites using three different DVVC methods. The geotechnical conditions were investigated by CPT soundings prior to and after treatment. Measurements of ground vibration were performed on two sites. During vibratory excitation, cylindrical shear waves were emitted along the vertically oscillating shaft, resulting in strong vertical ground vibrations.

Ground vibrations can be amplified by adjusting the resonance frequency of the vibrator–probe–soil system. The magnitude of vertical ground vibrations at a lateral distance of 2 m exceeds approximately 20 mm/s. Assuming that the compaction phase lasts 5 min, the number of vibration cycles exceeds 6000, which is significantly higher than that during a strong earthquake.

Field observations showed that liquefaction occurred at all sites in the uncompacted soil during the first pass of

compaction. However, during the second compaction pass, in-between the already treated soil columns, liquefaction did not occur.

A liquefaction analysis was performed, based on stress-adjusted CPT data, assuming a 7.5 magnitude earthquake. The analysis showed that liquefaction was likely to occur during vibratory excitation by the vertically oscillating compaction probe. As a result of DVVC compaction, the stress-adjusted cone resistance after compaction was typically higher than 10 MPa, which is beyond the cone resistance where liquefaction could be expected even during a very strong ($M=7.5$) earthquake.

CPT investigations showed that, in addition to the increase in cone resistance, the sleeve resistance also increased significantly. The increase in sleeve resistance reflects the change in horizontal effective stress and thus the overconsolidation ratio. This aspect is potentially of great significance for liquefaction analyses.

DVVC coupled with vibration measurements can be used as a full-scale liquefaction testing device in soil deposits down to about 15 m depth. DVVC will induce liquefaction in liquefiable sand and, conversely, indicate sand that is not liquefiable and, thus, determine whether or not a site is susceptible to liquefaction and if compaction is necessary.

A concept of full-scale liquefaction testing is outlined which can be readily used in combination with the critical strain method proposed by Dobry *et al.* (1982). Such tests can be used to verify in the field whether – and at which strain level – liquefaction occurs or not in a soil cylinder surrounding the compaction probe.

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