Verifying liquefaction remediation beneath an earth dam using SPT and CPT based methods☆

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Abstract
This paper presents a case study of liquefaction potential assessment for the foundation of an earth dam in Tunisia. An emphasis was made on the exploration of geotechnical conditions and the interpretation of field tests (SPT and CPT) and the results were collected before and after soil densification using the vibrocompaction technique. The assessment of soil liquefaction triggering was made using deterministic and probabilistic simplified procedures. The results indicate that before vibrocompaction the studied area was prone to the liquefaction hazard. However, after vibrocompaction a significant improvement of the soil resistance reduced the liquefaction potential of the sandy foundation soil. The SPT resistance values increased on average from 12 to 25 blow counts/0.3 m, and the CPT resistance increased on average from 8 MPa to 14 MPa. Before vibrocompaction, the factor of safety (FS) against liquefaction fell below 1.0, which means that the soil is susceptible for liquefaction. After vibrocompaction the values of FS exceed the unit which justified the liquefaction mitigation efforts in dam foundation.

1. Introduction
Soil liquefaction is a hazard for structures founded on sandy soil and silty soil and located in seismically active regions. Liquefaction-related damages to civil infrastructure were observed after the 1964 Niigata, Japan earthquake which helped to identify liquefaction as a major problem in earthquake engineering. The development of liquefaction engineering procedures has involved the synthesis of theoretical in situ-based empirical correlations over the last several decades. The simplified procedure was created initially based on extensive laboratory studies of the behaviour of soils subjected to cyclic loading and was later confirmed and supplemented with field case histories. As more and more case histories of soil liquefaction at sites with in situ testing became available, the simplified procedure was updated accordingly and become known in its present form. A number of simplified methods based on in situ tests, such as those developed by Seed et al. [20], Robertson and Campanella [19], Olsen [24], Youd et al. [26], Cetin et al. [10], Moss et al. [17] and Boulanger and Idriss [6], have been suggested and follow the general approach of the simplified procedures. The essential component of the deterministic and probabilistic methods is to identify an appropriate means of measuring, or estimating, the soil’s resistance to liquefaction during seismic loading.

The present paper examines the liquefaction potential at the site of the Sidi El Barrak dam, a large hydraulic project in the north of Tunisia. The vibrocompaction technique was selected as the appropriate method of ground improvement work in order to reduce the liquefaction hazard under the dam foundation. SPT and CPT tests were performed before and after soil densification and the results are discussed herein.

2. Site conditions
Sidi El Barrak earth dam is situated in the extreme North Western coast of Tunisia (Fig. 1). The site of dam is located at 6.5 km from the Mediterranean Sea, 15 km from the Nefza region and 20 km North East of Tabarka city [23]. Total area of the watershed is about 896 km² and the reservoir level is equivalent to 29 m height. The total capacity of the reservoir is about 275 million cubic meters. The Sidi El Barrak dam provides irrigation water for fertile lands that extend over an area of 4000 ha. The heterogeneous dam soil foundation is predominantly composed of Mio-Plio-Quaternary alluvial sediments mainly sandy with shallow ground water (1.5–5 m below the ground surface) (Fig. 2a and b).
The alluvium of the bed river is mainly composed of gravels, unconsolidated sand and silty lenses. The left bank is dominated by eolian dunes. The bedrock is formed by an Oligocene sedimentary deposit composed of marl which can be found at shallow depth on the right side. The foundation is formed by the following units [1]:

- **Eolian grounds** composed of fine to medium yellowish to brown sand.
- **Current formations** are composed of sands with sandstone debris, heterogeneous coarse sand, homogeneous medium sand and sandy silt with sandstone debris.
- **Alluvial grounds** constitute the main deposits filling the valley. The first 15 to 20 m are dominated by the alluvial sands which are homogeneous, highly uniform and lenticular with yellow-brown color.
- **Clay grounds** consisting of black clay, silty sands, silt and dark gray sandy clay with some lenses of fine to medium sands.
- **Oligocene substratum** is mainly formed by greenish brown marls with intercalations of siliceous sandstone benches.

According to the Tunisian Central Bureau [13], ground motions recorded in the western north of Tunisia are characterized by a maximum peak ground acceleration (PGA) equal to 0.2 g, variable intensities of VII to VIII and a magnitude equal to 7.

The study area has been the subject of a geotechnical survey including field and laboratory tests. Indeed, sampling and testing were carried out during July, September, October and November 1990. The dam foundation was subdivided into 45 square meshes of 100 m on each side where numerous SPT and CPT tests have been performed along multiple lines. 141 SPT's were conducted side by side (about 1.5 to 3.0 m apart) at the site. The SPT procedure was performed in accordance with ASTM D1586-84. The sampler is driven into the soil using a safety hammer of 63.5 kg mass falling through a height of 750 mm at the rate of 30 blows per minute. However, no energy measurement was made. The energy ratio was estimated to be in the range of 40–75% since the used equipment consisted in safety hammer. Near the SPT locations, 172 CPT's were conducted using the procedure specified in ASTM D3441. During the test, data were recorded every 5 cm. The area of seismic cone is 10 cm², the apex angle of the cone is 60°.

In addition, two wells were installed in the left side and in the bed river of Sidi El Barrak dam. Fig. 3 presents the typical grain size envelopes for depths where the two wells are performed. The results show the abundance of the alluvial sands in the former zone and the dominance of the aeolian sands in the latter zone [12].

Liquefaction criteria were derived from several case historic studies. Such criteria provided a basis for partitioning the soils vulnerable to severe strength loss as a result of an earthquake shaking. According to the laboratory test results, it is clear that the liquefaction conditions of a sandy soil are met. Indeed, a determination of the soil characteristics using the Fig. 3 shows that the median diameter D₅₀ is in the range of 0.05–1.5 mm and the uniformity coefficient is less than 15. Therefore, there is liquefaction potential in the Sidi El Barrak dam foundation.

Two ground improvement methods were initially selected and then discussed: the vibrocompaction and dynamic compaction techniques. The fine soils located in the central part and the left side of the project site were found to be highly compressible which would significantly dampen the transmission of dynamic shear stresses to the underlying soils, thereby reducing the treatment effectiveness beneath the soft soils using the dynamic compaction method. Thus, vibrocompaction technique was deemed as the most effective and economical choice in order to obtain a minimum target relative density of 70%, to achieve low static settlements and ensure liquefaction resistance. The treatment of Sidi El Barrak foundation soil, at about 10 m depth, has been achieved in equilateral triangular zone of spacing 2.94 m (Fig. 4). Fig. 5 shows the location of zones where vibrocompaction took place.

3. Analysis of pre- and post-remediation of liquefaction potential using SPT-based methods

Since the geotechnical investigation indicated that the liquefaction risk is high and the predicted liquefaction-induced

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Fig. 1. Location and components of Sidi El Barrak dam project.
settlements are not tolerable, the entire dam foundation was treated by the vibrocompaction method. This technique was selected to mitigate the liquefaction hazard and the settlement effects on the dam foundation. The main purpose of vibrocompaction is to densify the in situ soil by insertion of a vibration into the ground. The applied vibration associated with a large addition of water and air through jets along the vibration probe. The action leads to rearrange in a denser state. In the case of Sidi El Barrak dam, the typical equipment used in the vibrocompaction method includes an electric vibrator and a water pump. The vibrating unit operates at frequency of 1800 cycles per minute and with amplitude of 12 mm. Besides, the vibrator weighs about 2600 kg. A strict quality control program pursued in the project (before and after vibrocompaction) has implemented the SPT test in some locations in the foundation (Fig. 5). The zones C2, C4 and D2 were selected as examples of boreholes that presented in situ data to assess the liquefaction potential of the dam foundation. The two zones C2 and D2 are located in the left side of the dam while the zone C4 is situated on the bed river. It should be noted that the difference between the selected zones consists mainly in the soil type of each zone as indicated on the grain size distribution (Fig. 3a and b). Full details and the obtained results in others zones are not presented here for brevity.

Using the SPT results, the screening evaluation of the liquefaction potential of the Sidi El Barrak dam foundation is made by determining the critical value of the standard penetration resistance, \( N_{\text{crit}} \), separating liquefiable from nonliquefiable conditions [11]:

\[
N_{\text{crit}} = N_{\text{ref}} \times [(1 + 0.125 \times (d_s - 3) + 0.05 \times (d_w - 2)]
\]

where \( d_s \) is the depth of the sandy layer (m); \( d_w \) is the depth below upper level of water table (m); \( N_{\text{ref}} \) is a function of the earthquake intensity (Table 1).

Fig. 6 illustrates the variation in depth of the corrected SPT blow count \( (N_1)_{60} \) and the critical penetration resistance, \( N_{\text{crit}} \), for different earthquake intensities in zones C2, C4 and D2 before and after the vibrocompaction technique. From this figure, the results show increased \( (N_1)_{60} \) values in looser soil layers when compared with results obtained in the corresponding material prior to vibrocompaction. In fact, the \( (N_1)_{60} \) values increased from an average of 21 to 46 blows /0.3 m in mesh C2 and from an average

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**Fig. 2.** (a) Geological longitudinal section of the river. (b) Geological cross section of dam foundation.
of 25 to 43 blows/0.3 m in mesh C4 and from an average of 15 to 35 blows/0.3 m in mesh D2. In addition, before vibrocompaction, the SPT borings data are plotted below the threshold curve and are so potentially liquefiable (Fig. 6a, c and e). After vibrocompaction, the SPT data has exceeded the threshold curve and are not expected to liquefy (Fig. 6b, d and f).

Seed and Idriss [21] proposed a stress-based procedure to analyze liquefaction risk of soils. This procedure is largely based on empirical observations of laboratory and field data. It has been continually refined as a result of newer studies and the increase in the number of liquefaction case histories. The Seed and Idriss approach requires an estimate of the seismic demand placed on a soil layer (expressed in term of cyclic stress ratio CSR), the capacity of soil to resist liquefaction (presented in term of cyclic resistance ratio CRR) and the factor of safety (FS).

The calculation of CSR originally defined by Seed and Idriss [21] is given by the following equation:

$$CSR = \frac{0.65}{C_2} \frac{\sigma_v}{\sigma_v^*} \times \frac{a_{max}}{g} \times r_d$$

where, $\sigma_v$ and $\sigma_v^*$ are total and effective vertical overburden stresses respectively, $a_{max}$ is the peak horizontal acceleration at ground surface generated by the earthquake, $g$ is the acceleration of gravity and $r_d$ is a stress reduction coefficient.

To address the limited amount of field liquefaction data available in the 1970s for developing the simplified approach, Seed and Idriss [22] compiled a sizable data base from sites where liquefaction did or did not occur during earthquakes. They introduced a correction factor called magnitude scaling factor (MSF) in order to adjust the CSR value to magnitudes smaller or larger than 7.5. Different correlations for MSF have been proposed. The bases of these relationships are given and discussed in NCEER (1997) and Youd et al. [25,26].

The cyclic resistance ratio (CRR) can be determined graphically from charts plotting the CSR as a function of in situ test data (SPT blow count or CPT tip cone resistance). A boundary curve giving reasonable separation of the liquefied point and non liquefied points defines the CRR [20].

The calculated CRR can be compared to CSR in order to determine the factor of safety (FS) as follows:

$$FS = \frac{CRR}{CSR}$$
If the factor of safety is less than one, i.e., if the loading exceeds the resistance, liquefaction is expected to be triggered.

In the present paper, a focus is made on the most widely accepted methods used for evaluating soil liquefaction resistance. The state-of-the-art of SPT is Cetin et al. [10] and Idriss and Boulanger (2008). The state-of-the-art of CPT is Moss et al. [17] and Idriss and Boulanger (2008).

For this comparative evaluation the Seed and Idriss simplified equation (Eq. (2)) will be used to calculate the earthquake demand. However, the various parameters (e.g. $r_d$, MSF, $C_N$, $K_s$) will be evaluated by utilizing the specific recommendation discussed by each of the four methods.

### 3.1. Cetin et al. [10] Simplified procedure

The Cetin et al. [10] method treats liquefaction resistance (CRR) probabilistically and can be formulated as:

$$\text{CRR} = \exp \left( \left( \frac{N_{1,60}(1 + 0.004 FC) - 39.53 \ln(M_w) - 3.71 \ln\left(\frac{\varphi}{\varphi - 1}\right) + 16.85 + 2.76^{\varphi - 1}(PL)}{13.31} \right)^{\frac{1}{C_16/C_17}} \right)$$

where $N_{1,60}$=corrected SPT resistance, FC=fines content (in percent), $M_w$=moment magnitude, $\varphi$=total vertical stress, $\varphi'$=initial vertical effective stress, $\varphi^{-1}$=the inverse of the standard
normal cumulative distribution function, \( PL = \text{probability of liquefaction} \). Alternatively, given a \( CSR \) the \( PL \) can be calculated using Eq. (5).

\[
PL = \phi \left( \frac{N_{1,60} \times (1 + 0.004FC) - 13.32 \ln(CSR_{eq}) - 29.53 \ln(M_w) - 3.71 \ln(\phi) + 0.05FC + 16.85)}{2.7} \right)
\]

(5)

where \( CSR_{eq} \) is the cyclic stress ratio which is not adjusted for magnitude or duration effects; \( Pa \) is atmospheric pressure.

The probabilistic liquefaction analysis can use the plot showing contours of liquefaction probability (for \( PL = 5, 20, 50, 80 \) and 95%) as a function of the equivalent uniform cyclic stress ratio (\( CSR_{eq} \)) and the corrected blow count for fine content \( N_{1,60,CS} \) [9]. The equivalent uniform \( CSR_{eq} \) must be adjusted for duration effects based on the magnitude correlated DWFM as:

\[
CSR_{eq}^* = \frac{CSR_{eq}}{DWM_F}
\]

(6)

In order to use this relationship deterministically rather than probabilistically, Cetin et al. [10] recommended the use of CRR values corresponding to probability of liquefaction equal to 15 and varying fines content (\( FC < 5\% \), \( FC = 15\% \) and \( FC > 35\% \)).

Figs. 7-9 represent the graphs of calculated \( CSR_{eq}^* \) and corresponding \( N_{1,60,CS} \) from Sidi El Barrak dam foundation (respectively in meshes C2, C4 and D2). From these figures, it was observed a significant change of \( N_{1,60,CS} \) values calculated after vibrocompaction in meshes C2, C4 and D2. In fact, the \( N_{1,60,CS} \) values increase from an average of 21 to 47 in mesh C2, from an average of 22 to 33 in mesh C4 and from an average of 18 to 26 in mesh D2. These graphs show that data points recorded before vibrocompaction fall to the left of the boundary curve for different probability values. Thus, the untreated horizons are classified as liquefiable soils. After vibrocompaction, the data points occupy the region of the plot where no liquefaction was observed. Accordingly, the dam foundation is now deemed not susceptible to liquefaction.

Figs. 10-12 show the Cetin et al. [10] chart used for evaluating liquefaction potential of Sidi El Barrak dam foundation from the SPT results as a function of \( CSR_{eq}^* \). It was observed that in meshes C2, C4 and D2 the unimproved points were located to the left to the liquefaction triggering threshold. The improved points were found on the right of the liquefaction threshold curves.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{fig7.png}
\caption{Fig. 7. Relationship between \( CSR_{eq}^* \) and \( N_{1,60,CS} \) in C2.}
\end{figure}

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{fig8.png}
\caption{Fig. 8. Relationship between \( CSR_{eq}^* \) and \( N_{1,60,CS} \) in C4.}
\end{figure}

3.2. Boulanger and Idriss [5] Simplified procedure

Boulanger and Idriss [6] recommended a deterministic correlation between the cyclic resistance ratio (CRR) and the SPT penetration resistance. The SPT case histories of liquefaction in sands, silty sands and sandy silts were replotted against their equivalent clean values (\( (N_1)_{60,CS} \)) with different ranges of fines content. The derived boundary lines or the cyclic resistance ratio CRR adjusted to \( M=7.5 \) and \( \sigma_v’=1 \) atm for cohesionless soils can be calculated on the basis of the \( (N_1)_{60,CS} \) values via the equation presented below:

\[
CRR_m = 7.5 = \exp \left( \frac{(N_1)_{60,CS}}{14.1} + \frac{(N_1)^2_{60,CS}}{126} - \frac{(N_1)^3_{60,CS}}{234} + \frac{(N_1)^4_{60,CS}}{254} - 2.8 \right)
\]

(7)

The parameters of \( r_d, MSF, K_r, C_v \) were modified and new correlations were presented [2]. The modified relations were then used to derive revised deterministic SPT and CPT-based liquefaction correlations. According to Boulanger & Idriss [6], the \( (N_1)_{60,CS} = 37.5 \) has been interpreted as the upper bound cutoff between liquefiable and non liquefiable soils. However, Cetin et al. [10] did not recommend a specific limiting upper value for \( N_{1,60,CS} \).
Figs. 14–16 show the resulting proposed deterministic relationship between cyclic stress ratio and fines corrected penetration resistance ($N_{1,60,CS}$) in three meshes (C2, C4 and D2) of the Sidi El Barrak dam foundation. The obtained results indicate that the compacted soils present high fines corrected penetration resistance $N_{1,60,CS}$ values. The average of $N_{1,60,CS}$ values increase from 11 to 41 in zone C2, from 11 to 27 in zone C4 and from 12 to 22 in zone D2. In these figures, the solid dots represent the pre-treatment point data for which liquefaction can be triggered, the post-treatment data represent non liquefaction cases.

Fig. 17 shows the FS profile calculated from the Boulanger and Idriss approach in zones C2, C4 and D2 before and after soil improvement. The FS profile obtained from the pre-treatment data are less than the critical value (FS = 1). So, the dam foundation is deemed prone to liquefaction during the design earthquake event. Nevertheless, the gaps in the critical value data represent soil layers that are not susceptible to liquefaction due to their densification by vibrocompaction.

4. Analysis of liquefaction potential of Sidi El Barrak dam foundation using CPT-based methods

The first CPT liquefaction correlations based directly on case histories were published by Zhou [27] using observations from the 1978 Tangshan earthquake. Zhou [27] (in [22]) identified the liquefaction potential with the formula:

$$q_{\text{crit}} = q_{\text{co}} \left[ \left( 1 - 0.065(z_w - 2) \right) \left( 1 - 0.05(z_s - 2) \right) \right]$$

where $q_{\text{crit}}$ is the critical resistance for liquefaction potential; $q_{\text{co}}$ is the static penetration resistance that depends on epicentral intensity of considered earthquake (Table 2); $z_w$ is the depth of water table level from ground surface (in meters); $z_s$ is the distance between water table level and point of measurement (in meters).

The CPT data collected before and after the soil improvement of the Sidi El Barrak dam foundation (in the meshes C2, C4 and F4) and the threshold curves given by Zhou [27] for different earthquake intensities are illustrated in Figs. 18–23. Before vibrocompaction, the measured values of $q_e$ are generally lower than the critical resistance values $q_{\text{crit}}$, showing vulnerability of the dam.
foundation to liquefaction. The $q_c$ values in the compacted sand increase significantly due to the soil consolidation and rearrangement of particles after soil densification. However, at some depths, there is practically no change in $q_c$, indicating that ground improvement is not effective in those layers. The possible reason of this effect is that the maximum depth of the vibrocompaction is less than 12 m.


The CPT-based deterministic method can be modified to account for the effect of nonplastic fines content on the liquefaction resistance by using an approach similar to the one used for the SPT-based correlation [4]. Accordingly, the CRR relationship

![Figure 13. FS profile (using PL=15%) before and after vibrocompaction in the zones C2, C4 and D2.](image)

![Figure 14. Relationship between CSR and $(N_1)_{60cs}$ in mesh C2.](image)

![Figure 15. Relationship between CSR and $(N_1)_{60cs}$ in mesh C4.](image)

![Figure 16. Relationship between CSR and $(N_1)_{60cs}$ in mesh D2.](image)
derived from the liquefaction correlations by Idriss and Boulanger is illustrated by the following equation:

\[
CRR_M = 7.5 = -\exp\left(\frac{q_{c1NCS}}{540} + \left(\frac{q_{c1NCS}}{67}\right)^2 + \left(\frac{q_{c1NCS}}{80}\right)^3 + \left(\frac{q_{c1NCS}}{114}\right)^4 - 3\right)
\]

where \(q_{c1NCS}\) is the fines content corrected penetration resistance.

Figs. 24–26 show calculated cyclic stress ratio plotted as a function of normalized and corrected CPT resistance cone \(q_{c1N}\) from Sidi El Barrak site (in meshes C2, C4 and F4).

The pre-treatment data points are plotted below the boundary curve which indicates that the soils in zone C2, C4 and F4 are susceptible to the cyclic liquefaction. However, the post-treatment data fall above the boundary curve, in the non-liquefaction zone.

In the study case, the results of the liquefaction analysis before and after soil compaction, expressed as the profile of safety obtained from the Idriss and Boulanger (2008) method, are reported respectively in Figs. 27 and 28. Before improvement conditions, Fig. 27 shows that the factor of safety drops below 1 which indicates that the investigated layers in zones C2, C4 and F4 are susceptible to liquefaction at the expected future earthquake. In Fig. 28, after improvement, the factor of safety against liquefaction exceeds 1. Hence, the saturated sandy layers are not

<table>
<thead>
<tr>
<th>Intensity</th>
<th>(q_{c})</th>
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<tbody>
<tr>
<td>VII</td>
<td>82</td>
</tr>
<tr>
<td>VIII</td>
<td>117</td>
</tr>
<tr>
<td>IX</td>
<td>182</td>
</tr>
</tbody>
</table>

Table 2: \(q_{c}\) as a function of the earthquake intensity and PGA.

Fig. 17. FS profile before and after vibrocompaction in the zones C2, C4 and D2.

Fig. 18. CPT data before vibrocompaction in mesh C2.

Fig. 19. CPT data before vibrocompaction in mesh C4.
vulnerable to liquefaction menace under the design earthquake. The improvement process was effective in eliminating liquefaction potential in the dam site.

4.2. Moss et al. [18] Simplified procedure

Based on the compilation, processing and evaluation of an extensive database of field observations of liquefaction and non liquefaction, Moss et al. [17] employed a Bayesian updating methodology to develop probabilistic correlations for liquefaction
initiation for $M=7.5$ and $\sigma'_v=100$ kPa. The resultant correlation for liquefaction probability is given by:

$$PL = \phi\left(-\frac{(q_{c,1}^{0.105} + q_{c,1}(0.11 \times R_f) + c(1 + 0.85 \times R_f) - 7.177\ln(CSR) - 0.848\ln(M_w) - 0.002\ln(\sigma'_v) - 20.923}{1.32}\right)$$

(10)

where $q_{c,1}$ = normalized tip resistance (MPa); $R_f$ = friction ratio (percent); $c$ = normalization exponent; CSR = equivalent uniform cyclic stress ratio; PL = cumulative normal distribution.

The cyclic resistance ratio for a given probability of liquefaction is expressed by:

$$CRR = \left(\frac{(q_{c,1}^{0.105} + q_{c,1}(0.11 \times R_f) + (0.001 \times R_f) + c(1 + 0.85 \times R_f) - 0.848\ln(M_w) - 0.002\ln(\sigma'_v) + 1.632 \times \phi^{-1}(PL)}{7.177}\right)$$

(11)

Differences between probabilistic [17] and deterministic [6] liquefaction triggering correlations are partly due to the use of statistical analyses and regression approach as well as to differences in their underlying databases and relevant parameters ($r_d$, correction factors…).

Figs. 29–31 illustrate the plot of contours of probability of liquefaction (PL = 5%, 20%, 50%, 80%, 95%) [15]. It should be noted that a probability of liquefaction PL = 15% was selected to represent the deterministic liquefaction approach.

Fig. 27 presents the FS profiles in meshes C2, C4 and F4 when the case of the dam foundation is re-analyzed using the Moss et al. [17] simplified procedure. Before vibrocompaction, the soils appear liquefiable with factor of safety smaller than the unit. After vibrocompaction, the derived profile of FS shows that the majority of soil layers have a factor of safety greater than 1.
5. Comparison liquefaction triggering procedures

Regarding evaluation of liquefaction triggering via simplified procedures, the three variables most responsible for difference between the Cetin et al. [10] and Boulanger and Idriss [6] procedure are $r_d$, $C_N$ and $K_s$. In term of earthquake loading, the various $r_d$ relationship tend to produce CSR values which are lowest for Cetin et al. [10] and highest for Boulanger and Idriss [6]. In term of soil resistance, the various CN relationships tend to yield $(N_{1,60})$ values that are highest for Boulanger and Idriss [6].

Fig. 33 illustrates a comparison of the proposed probabilistic curves with the clean sand curves from Boulanger and Idriss [6] in mesh C2 before and after soil improvement. This comparison shows general agreement between the proposed probabilistic curves and the deterministic correlations. In fact, a major part of the pre-treatment data is located to the left of the deterministic and probabilistic boundary curves and the post-treatment data fall on the right of the boundary curves. However, it is particularly important to note that some $N_{1,60,CS}$ values obtained from Cetin et al. [10] method lie above the dashed threshold line of Boulanger and Idriss [6] and they will be expected to liquefy. Furthermore, a comparison between CPT-probabilistic [18] and deterministic [6] correlations shows that the former yields lowest corrected CPT tip resistance values while the latter provide the highest $q_{c,1}$ values (Figs. 24–26 and Figs. 29–31). It could simply mean that the overburden normalization factor for penetration resistances $C_p$ appears to have a considerable influence on the agreement or disagreement of the simplified procedures.
Fig. 34 presents a comparison between deterministic methods of Corte [11] and Cetin et al. [10] (for PL = 15%) using Pre-treatment SPT data in mesh C2. This figure shows that $\left(N_{15}\right)_{ao}$ values obtained from Cetin et al. [10] are almost located below the boundary curve. In Fig. 35, the post-treatment data produced by the Cetin et al. [10] fall above the threshold liquefaction triggering which indicates that the treated soils in mesh C2 are non liquefiable. Thus, this comparison revealed that the SPT-deterministic liquefaction methods are similar.

Fig. 36 provides a comparison between the Moss et al. [17] and Zhou [27] procedures. It is noted that the majority of $q_{c,1,mod}$ points calculated from Moss et al. [17] are plotted on the left of the boundary curve suggested by Zhou [27]. However, some $q_{c,1,mod}$ data are situated on the right of the recommended boundary curve and they are so classified as nonliquefiable. It appears that this difference in liquefaction triggering is primarily driven by variations in the cone resistance $q_c$ correction factor $C_q$. Indeed, Zhou did not recommend any correction of the $q_c$ values whereas Moss et al. [17] have defined a normalization exponent involving an iterative procedure.

6. Conclusion

Based on the results of the presented case study, the SPT and CPT tests are shown to be an effective tool for characterizing the liquefaction potential of the dam site before and after the vibrocompaction ground improvement. Based on this liquefaction analysis, the following conclusions are reached:

1. The effectiveness of vibrocompaction soil densification in reducing the liquefaction potential, as mentioned in literature, is confirmed in this case study. The factor of safety against liquefaction is obtained from SPT and CPT-based simplified procedures. The pre-treatment subsurface penetration data plotted to the left of the recommended deterministic and probabilistic liquefaction triggering threshold, whereas the post-treatment data plotted on the right of the boundary curve. The results show that the undensified alluvial sands of foundation were prone to liquefaction hazard ($FS < 1.0$). However, after vibrocompaction, the dam foundation was deemed not susceptible to liquefaction ($FS > 1.0$).

2. The liquefaction evaluation results based on the SPT data are similar to those based on the CPT data. A comparison shows general agreement between the deterministic and probabilistic correlations. However, Cetin et al., [10] procedure results in lower FS. The Moss et al. [17] and the Idriss and Boulanger (2008) results are comparable. Differences between probabilistic and deterministic liquefaction triggering correlations are partly due to the use of statistical analyses and regression approach as well as to differences in their underlying databases and relevant parameters ($r_d$, correction factors...).

The use of laboratory tests to predict the liquefaction potential of the dam is a considerable interest to be carried out by the Authors. A further study based on a comparison between laboratory and in situ tests results appears to be opportune.
Fig. 33. Comparison of Cetin et al [10] and Boulanger and Idriss [5] methods.

Fig. 34. Comparison of deterministic correlations of Corte [11] and Cetin et al [10] (PL=15%) in mesh C2 (before vibrocompaction).

Fig. 35. Comparison of deterministic correlations of Corte [11] and Cetin et al [10] (for PL=15%) in mesh C2 (After vibrocompaction).
References


Fig. 36. Comparison of deterministic correlations of Zhou [27] and Moss et al [18] (for PL=15%) in mesh C2 (Before vibrocompaction).