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Evaluation of Liquefaction Potential of an Earth Dam Foundation Using In Situ Tests

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EVALUATION OF LIQUEFACTION POTENTIAL OF AN EARTH DAM FOUNDATION USING IN SITU TESTS

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ABSTRACT

This paper presents a case study of liquefaction potential assessment carried out under an earth dam foundation in Tunisia. An emphasis was made on the exploration of geotechnical conditions and the interpretation of field tests results collected before and after soil densification using the vibrocompaction technique. The assessment of soil liquefaction susceptibility was made using deterministic and probabilistic simplified procedures developed from several case histories. Conclusively, the obtained results show that before vibrocompaction the soil was prone to the liquefaction hazard. However, after vibrocompaction, a significant improvement of the soil resistance reduces the liquefaction potential of the sandy foundation. Indeed, before vibrocompaction, the factor of safety (FS) drops below 1 which means that the soil is susceptible for liquefaction. However, after vibrocompaction, the values of FS exceed the unit which justify the absence of liquefaction hazard in the dam foundation.

In addition, before soil densification, the liquefaction evaluation using CPT-data shows probabilities values over 65 % which correspond to the classes of ‘’very likely’’ and ‘’Almost certain that will be liquefy’’ in the field case histories classification. The treated site presents low probability of liquefaction (less than 35%) indicating a low likelihood of liquefaction of the dam foundation.

Key Words: loose sand, liquefaction, standard penetration test, cone penetration test, vibrocompaction, Probability, Liquefaction Potential Index, field case histories.

INTRODUCTION

Liquefaction is a major concern for structures made with or on sandy soils. It is commonly observed in loose and saturated deposits of cohesionless soils subjected to large magnitude earthquakes. Since Niigata earthquake in 1964, researchers (Robertson &Campanella 1985; Shibata & Teparaska 1988; Olson et al. 1998; Robertson & Wride 1998; Juang et al. 2002; Boulanger & Idriss 2004) have developed a variety of simplified procedures using field investigations and laboratory tests in order to predict the liquefaction occurrence.

In the north littoral of Tunisia, the seismic character of the area and the sandy nature of soils might induce the soil liquefaction phenomenon. In this regards, the Sidi El Barrak earth dam, a large hydraulic project, provides an interesting case for assessing the liquefaction susceptibility of soils and evaluating the foundation stability. A ground improvement by vibrocompaction was done to mitigate the liquefaction hazard under the dam foundation.

This paper presents, first, an overview about Sidi El Barrak dam and its soil of foundation. Then, from the results of SPT and CPT tests conducted before and after vibrocompaction the liquefaction hazard is predicted and discussed.

SITE OF PROJECT DAM

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 (Technical document, 1990). Total area of dam is 4,000 hectares and the reservoir level is equivalent to 29 m height. Total capacity of reservoir is about 275 Million cubic meters.

The heterogenous foundation of dam is predominantly composed by sandy formations. The latter of Quaternaries, Neogene’s and Paleogene age consist in alluvial sand and eolian dunes. The rigid stratum level is composed by gneiss and marlstone which are apparent at the right side (fig. 2).

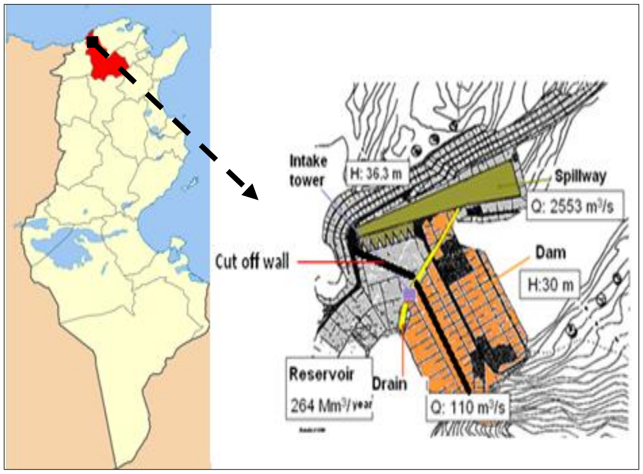

Fig. 1. Location and Components of Sidi El Barrak dam

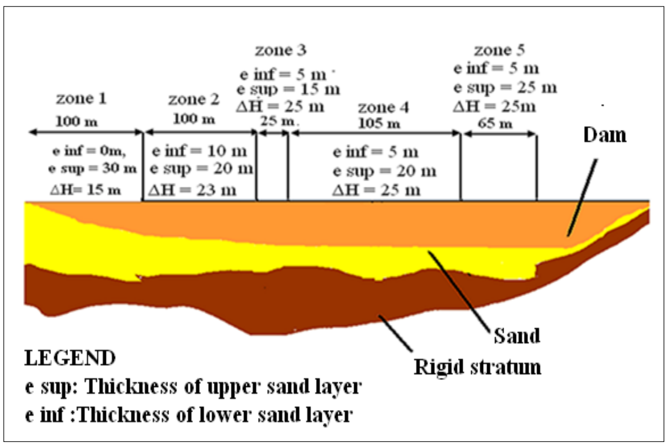

Fig. 2. Geological section of the dam site

The study area has been the subject of a geotechnical survey including field and laboratory tests. Indeed, two wells were executed respectively in the left side and the bed river of Sidi El Barrak dam. The results show the abundance of the alluvial sands in the former zone and the dominance of the eolian sands in latter zone (fig. 3a and fig. 3b). The water table level is generally about 5 m below ground surface in the two zones.

Furthermore, some liquefaction criteria were derived from several case histories data. Such criteria provided a basis for partitioning the soils vulnerable to severe strength loss as a result of an earthquake shaking. For instance, a sandy soil may be susceptible to liquefaction if it has the following characteristics:

-The degree of saturation is equal to 100%;
-The median diameter D_{50} is in the range of 0.05 mm to 1.5 mm;
-The uniformity coefficient is less than 15.

According to the laboratory test results (table 1), it is clear that the previous condition of the liquefaction criteria are met. Therefore, the liquefaction hazard may occur in the Sidi El Barrak dam foundation.

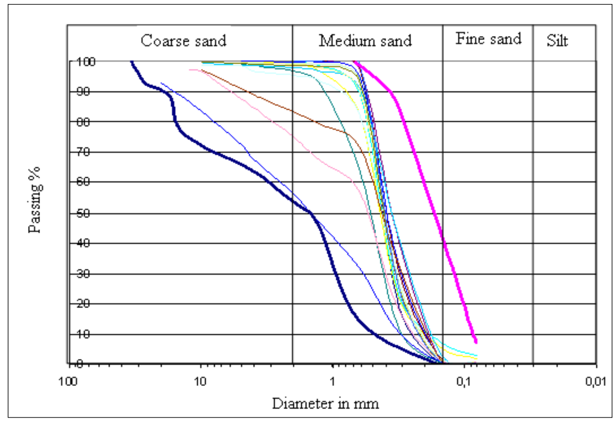

Fig. 3a. Grain-size distribution of soil in the left bank

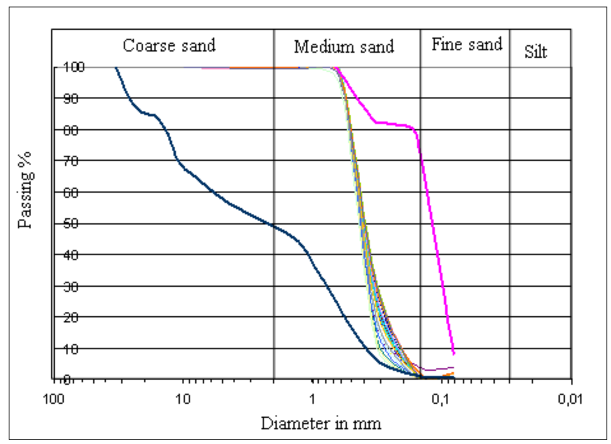

Fig. 3b. Grain-size distribution of soil in the bed rive

Table 1. Liquefaction susceptibility of Sidi El Barrak foundation

Zone	Curve	D_{60}	D_{50}	D_{10}	C_u
Left side	Upper curve	0.19	0.14	0.8	2.375
	Lower	3.00	1.30	0.40	7.50
River bed	Upper	0.16	0.13	0.080	2.00
	Lower	5.00	1.40	0.38	13.60

Consequently, the soil improvement using vibrocompaction is crucial for increasing the relative density of soils and reduces the liquefaction risk. The treatment of Sidi El Barrak soil, along 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.

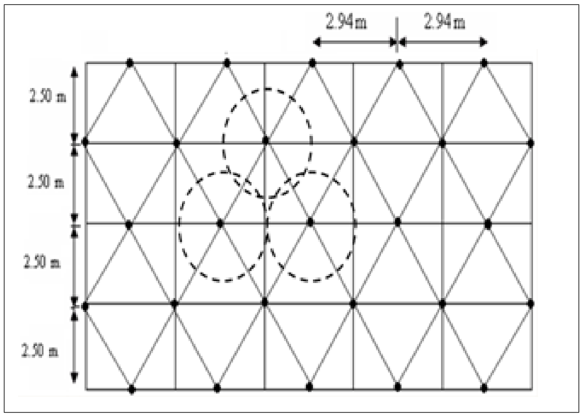

Fig. 4. Triangular mesh treated by vibrocompaction technique

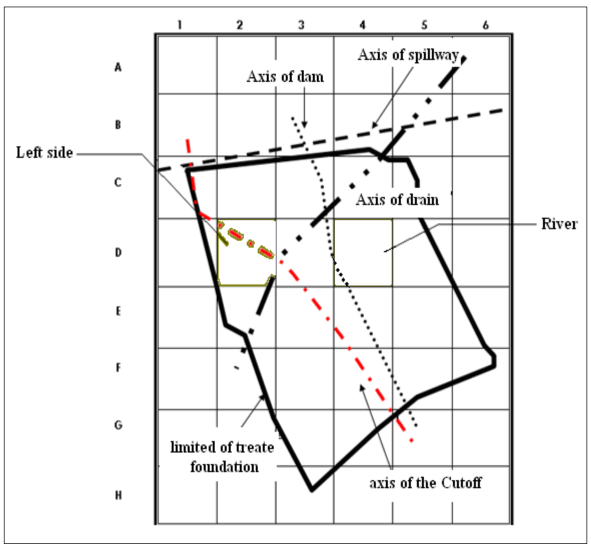

Fig. 5. Vibrocompacted zone

SPT AND CPT BASED ANALYSIS OF LIQUEFACTION POTENTIAL OF SIDI EL BARRAK DAM FOUNDATION

The SPT and CPT tests remain the most commonly in-situ test for sites investigation. Many empirical relations have been established between the SPT or CPT data (the SPT bow count or the cone penetration resistance, respectively) and other engineering properties of soils in order to understand and evaluate the liquefaction potential.

Evaluating the liquefaction potential of the Sidi El Barrak dam foundation is made by adopting the reference equation which allows the prediction of corrected number cycles as expressed by Trifunac & Brady(1975) and reported by Seed et al (1983):

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

where d_s is the depth of the sandy layer (m) d_w is the depth below upper level of water table (m) ;N_{ref} is the number of cycles for penetration equals to 30 cm, depending on the earthquake magnitude.

Figures 6 and 7 illustrate the variation in depth of the corrected SPT blow count $(N_1)_{60}$ (correction factors will be discussed later in this section) and N_{crit} for different earthquake intensities in zone C2. Indeed, the plots show three curves that represent the VII, VIII and IX input intensities and that are used to evaluate whether a sample would liquefy or not. Before vibrocompaction, the SPT borings data are plotted below the threshold curve and are so potentially liquefiable (fig. 6). After vibrocompaction, the corrected SPT blow count increased and reached 90 blows/0.3cm. The SPT data has exceeded the threshold curve and are not expected to liquefy (fig.7).

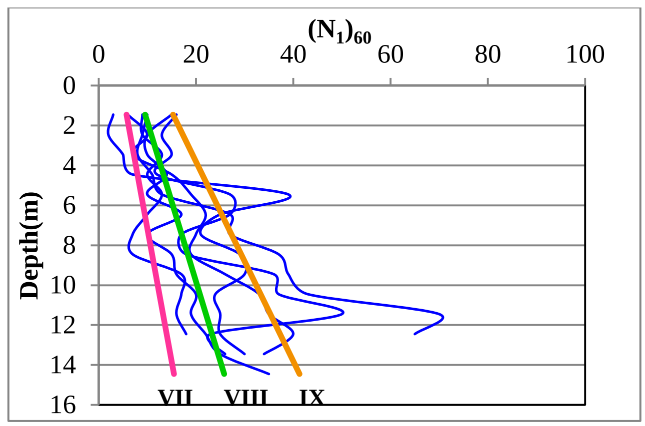

Fig. 6. Pre treatment corrected values in zone C2

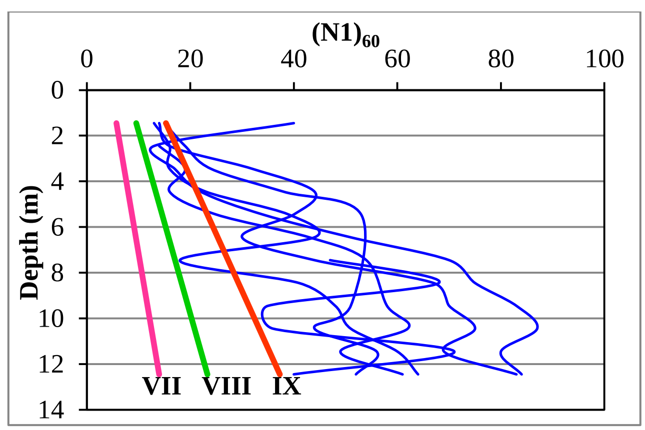

Fig. 7. Post treatment corrected values in zone C2

Furthermore, based on the CPT results, Zhou, 1980 (in Seed et al, 1983) had considered such data to identify the liquefaction potential from the formula:

$$q_{crit}=q_{c0}[(1-0.065(z_w-2)][1-0.005(z_s-2)] \quad (2)$$

Where q_{crit} is the critical resistance under which liquefaction risk is potential; q_{c0} is the static penetration resistance that depends on epicentral intensity of considered earthquake ; Z_w is th 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 and the threshold curves given by Zhou (1980) for peak ground accelerations of 0.15g and 0.2g are illustrated in figures 8 and 9. It can be seen the existence of liquefaction risk for earthquake with magnitude 0.2 g and, with more less influence for earthquake with magnitude 0.15 g.

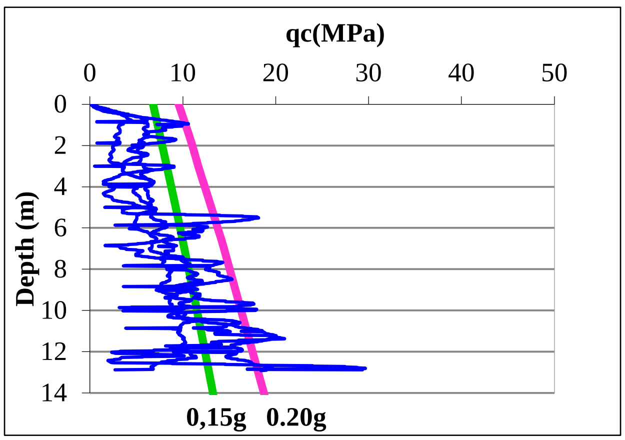

Fig. 7. Recorded CPT data before vibrocompaction in mesh C2

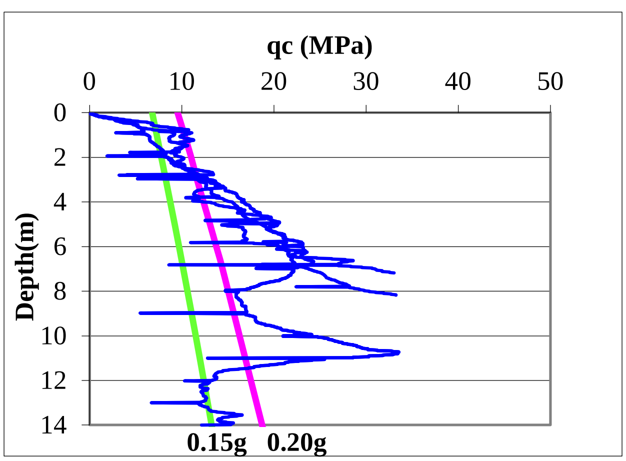

Fig. 8 . Recorded CPT data after vibrocompaction in mesh C2

Seed and Idriss (1971) outlined a simplified procedure to evaluate the liquefaction resistance of sandy soils using the relative density and the shear stresses induced by earthquake loading. In a later up date, using liquefaction case histories, Seed et al (1985) proposed a boundary curve which separates sites where liquefaction effects were or were not observed due to an earthquake magnitude of 7.5. This approach requires an estimate of the seismic demand placed on a soil layer, expressed in term of Cyclic Stress Ratio (CSR) (Youd et al., 1996). They formulated the simplified equation to calculate the CSR as following:

$$\mathrm{CSR} = 0.65 * \frac{\sigma_v}{\sigma'_v} * \frac{a_{max}}{g} * r_d \quad (3)$$

where, σ_v and σ'_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.

Because of the limited amount of field liquefaction data available in 1970s, for developing the simplified approach, Seed and Idriss (1982) compiled a sizable data base from sites where liquefaction did or did not occur during earthquake with magnitude near 7.5. Consequently, 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. (2001). Seed defines the variable MSF by the following equation:

$$\mathrm{MSF} = \left(\frac{M_W}{7.5}\right)^n \quad (4)$$

Where M_w is the moment magnitude and n is an exponent. In the present study, n is set to be equal to -2.56.

Seed et al (1982) suggested an empirical correlation between the CSR and the corrected SPT blow count $(N_1)_{60}$ in order to represent the soil liquefaction resistance. The $(N_1)_{60}$ is defined as the SPT blow count normalized to an overburden pressure of 100 kPa and to an energy level equal to 60% of the theoretical free-fall hammer energy applied to the drill. This correlation were developed for granular soils with the fines contents of 5% or less, 15%, and 35%.

Figures 9 and 10 represent the graphs of calculated cyclic stress ratio and corresponding $(N_1)_{60}$ from Sidi El Barrak dam foundation (respectively in meshes C2 and E3). The boundary line, expressed in term of the cyclic resistance ratio or the liquefaction resistance of soils, is positioned to separate region with data indicative of liquefaction from region with data indicative of non liquefaction. Those graphs show that data points recorded before vibrocompaction (solid triangles) fall to the left of the boundary curve (FC ≤ 5%). 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 not exposed to the liquefaction risk.

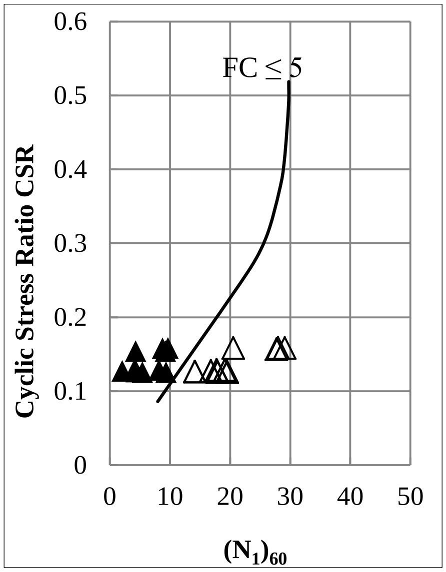

Fig. 9. Relationship between CSR and $(N_1)_{60}$ in C2

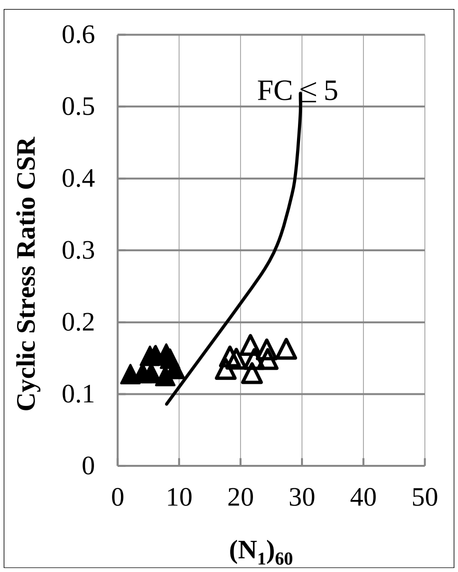

Fig. 10. Relationship between CSR and $(N_1)_{60}$ in E3

Besides, the increased field performance data have become available at liquefaction sites investigated with CPT tests. These data have facilitated the development of CPT-based liquefaction resistance correlations. In fact, Robertson and Campanella (1985) proposed a chart for estimating CRR from corrected CPT penetration resistance (q_{c1}) based on Seed et al.(1985) SPT chart and SPT-CPT conversions. This correlation has been developed using field observations collected from sites having the following conditions: level to gently sloping, terrain underlain by Holocene alluvial or fluvial sediment, depth range from 1 to 15m and magnitude M_W=7.5. The CPT procedure requires a normalization of tip resistance using equations 5 and 6. This transformation leads to a normalized, dimensionless cone penetration resistance (q_{c1N}).

$$q_{c1N} = C_Q \left(\frac{q_c}{P_{a2}}\right) \tag{5}$$

$$C_Q = \left(\frac{P_a}{\sigma'_v}\right)^n \tag{6}$$

Where q_c is the measured cone tip penetration resistance; C_Q is a correction for overburden stress; the exponent n is typically equal to 0.5; P_a is a reference pressure in the same unit as σ'_v (i.e., P_a = 100kPa if σ'_v is in kPa); P_{a2} is a reference pressure in the same unit as q_c (i.e., P_{a2} = 0.1MPa if q_c is in MPa). A maximum value of C_Q =2 is generally applied to CPT data at shallow depths.

Figures 11 and 12 show calculated cyclic stress ratio plotted as a function of corrected and normalized CPT resistance cone q_{c1N} from Sidi El Barrak site (in meshes C2 and F4).

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

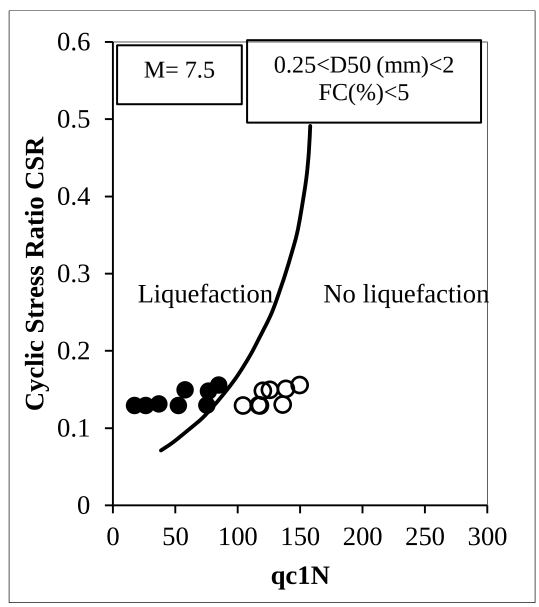

Fig. 11. CSR as a function of q_{c1N} in mesh C2

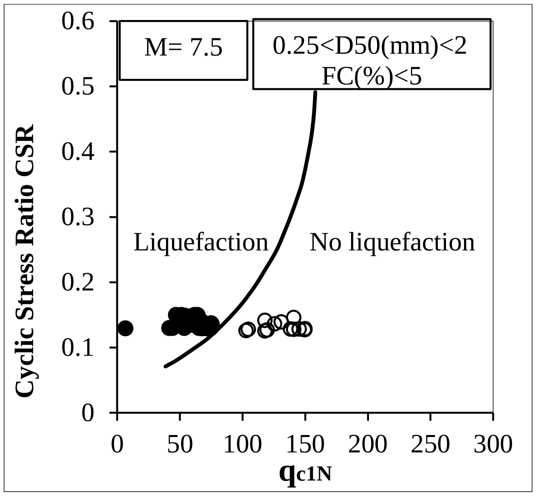

Fig. 12. N as a function of q_{c1N} in mesh F4

On the other hand, it is well-known that all simplified methods that follow the general stress-based approach pioneered by Seed and al require the determination of the cyclic stress ratio CSR and the cyclic resistance ratio CRR. As noted previously, CSR (equation 3) represents the seismic load imparted to the soil whereas CRR represents the capacity of soil to resist to initiation of liquefaction. The results of this deterministic approach are usually presented in a factor of safety (F_S), defined as the ratio of CRR over CSR. In theory, liquefaction is predicted to occur if $F_S \leq 1$, and no liquefaction is predicted if $F_S > 1$.

The liquefaction resistance CRR is generally evaluated from in situ tests. The 1996 NCEER the 1998 NCEER/NSF workshops reviewed the state of art of the Seed et al method and recommended revised criteria for evaluating CRR from SPT and CPT results. According to the various methods, CRR is evaluated graphically by use of charts. The boundary curve giving a reasonable separation of the liquefied and non liquefied points defines the CRR. Then, several authors have established empirical correlations for evaluating liquefaction potential. For example, based on the SPT data, the simplified curve in figures 11 and 12 is given by the following equation:

$$CRR_{7.5} = \frac{a + cx + ex^2 + gx^3}{1 + bx + dx^2 + fx^3 + hx^4} \qquad (7)$$

where $CRR_{7.5}$ is the cyclic resistance ratio for magnitude equal to 7.5; x = (N1) ; a = 0.048 ;b= -0.1248 ;c= -0.004721 ;d = 0.009578 ;e = 0.0006136 ;f = -0.0003285 ; g= -1.673 E-05 ;h = 3.714 E-06.

Figure 13 shows the profile of factor of safety obtained from the Blake method in the Sidi El Barrak dam foundation (mesh C2). Before vibrocompaction, the FS profile indicates that the study site has a high liquefaction potential, as almost all of the calculated FS are less than 1. After vibrocompaction, the FS values are greater than 1 which assumes that no liquefaction occurs in the improved soil body under the design seismic loading.

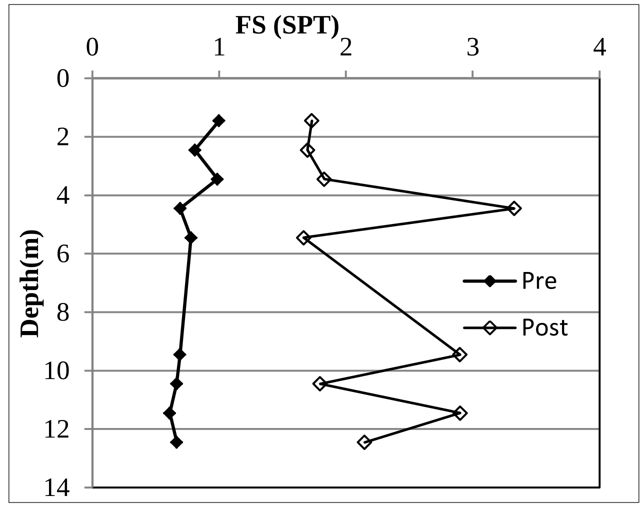

Fig. 13. Fs profile before vibrocompaction in the zone C2

Deterministic approach includes procedures based on the CPT data such as the Robertson method. In fact, the method proposed by Robertson & Wride (1998) provides an integrated procedure for evaluating the cyclic resistance of saturated sandy soils.

The measured penetration resistance can be corrected to an equivalent clean sand value:

$$(q_{c1N})_{cs} = K_c * q_{c1N} \qquad (8)$$

where K_c is a correction factor that is a function of the grain characteristics of the soil, q_{c1N} is the normalized penetration resistance obtained as described previously by using equation 6.

Then using the equivalent clean sand normalized penetration resistance $(q_{c1N})_{cs}$, the CRR (for M_w=7.5) can be estimated by the following equations:

If $(q_{c1N})_{cs} < 50$:

$$CRR = 0.833 \times \left[\frac{(qc1N)_{cs})}{1000}\right] + 0.05 \qquad (9)$$

If $50 < (q_{c1N})_{cs} < 160$

$$CRR = 93 \times \left[\frac{(qc1N)_{cs})}{1000}\right]^3 + 0.08 \qquad (10)$$

Figures 14 and 15 show the FS profile calculated from the Robertson & Wride approach in zone C2before 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 may be prone to liquefaction during an 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.

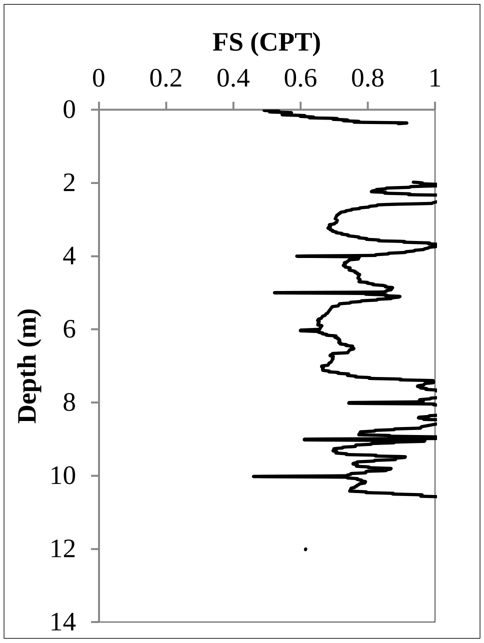

Figure 14. FS profile in zone C2 before vibrocompaction

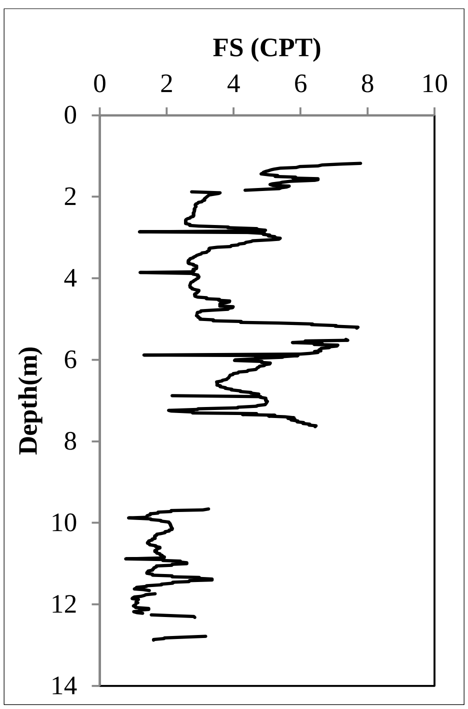

Figure 15. FS profile in zone C2 after vibrocompaction

The deterministic liquefaction evaluation method can only answer whether the soil liquefy (FS≥1) or not (FS<1). Thus, the probabilistic approach is increasingly used for quantifying the liquefaction hazard of the various verticals and for drawing up liquefaction potential maps. Actually, researchers suggested that any deterministic method must be calibrated so that the meaning of the calculated FS is understood in terms of likelihood or probability of liquefaction. For example, based on both logistic regression and Bayesian mapping approaches, the Robertson method has been calibrated by Juang and Jiang (2000) and the result was presented in the following mapping function:

$$P_L=\frac{1}{1+\left(\frac{FS}{A}\right)^{B}} \qquad (11)$$

Where the coefficients A= 1.0 and B=3.3.
After Chen and Juang (2000) the likelihood of liquefaction can be interpreted using the calculated *PL* values in Table 2.

Table 2. Classification of probability of liquefaction

Probability	Likelihood of liquefaction
0.85≤ PL<1	Almost certain that will be liquefy
0.65≤ PL<0.85	Very likely
0.35≤ PL<0.65	Liquefaction/ non liquefaction is equally likely
0.15≤ PL<0.35	Unlikely
0.00≤ PL<0.15	Almost certain will not liquefy

CPT data at the mesh C2 of the dam foundation are used as example to represent the profiles of the probability of liquefaction (P_L) obtained from the Robertson method described previously (fig. 16 and fig. 17). Before vibrocompaction (fig. 16), the profiles suggest that the calculated probabilities are high, ranging from 0.4 to 0.8, which fall into the classes of ''very likely'' and ''Almost certain that will be liquefy'' in the Juang and Chen classification given in table 3. After vibrocompaction (fig . 17), the profiles show low likelihood of liquefaction of soil.

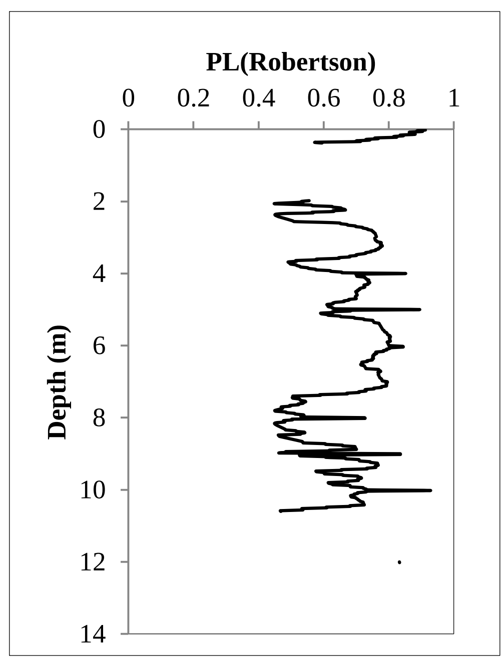

Fig. 16. Profile of P_L in zone C2 before vibrocompaction

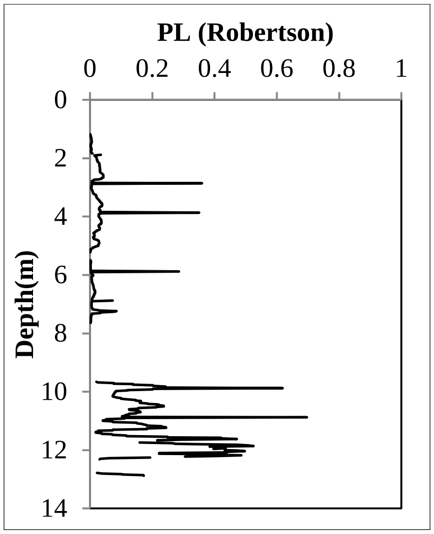

Fig. 17. Profile of P_L in zone C2 after vibrocompaction

Additionally, a probabilistic methodology, based on the use of the liquefied potential index I_L, was applied in order to evaluate the liquefaction hazard of the various explored verticals. The LPI was originally developed by Iwasaki et al (1982) to estimate liquefaction potential causing foundation damage (Holzer et al., 2003). The advantage of the index is that it attempted to predict liquefaction severity of the entire soil column whereas the simplified procedure originated by Seed et al (1971) predicts the liquefaction potential of a soil element.
Iwasaki et al (1982) introduce the following form for the liquefaction potential index as given by the equation (12) (Lee et al., 2003):

$$IL = \int_0^{20} F\, w(z)\, dz \qquad (12)$$

Where the variable F is defined as follows: F = 1 – FS for FS < 1; and F=0 for FS > 1. The weighting factor w (z) = 10 - 0.5z, z = depth (m).

Based on cases studied in Japan, Iwasaki et al (1982) provided the following liquefaction risk criteria, referred to herein as the Iwasaki criteria (Juang et al., 2006):
I_L= 0, the liquefaction failure is extremely low;
$0 < I_L \leq 5$, the liquefaction failure is low;
$5 < I_L \leq 10$, the liquefaction failure is high;
$10 < I_L \leq 15$, the liquefaction failure is low;
$I_L > 15$, the liquefaction failure is extremely high;

In the present study, the Liquefaction Potential Index I_L values were computed using the FS profiles obtained from the Robertson method.

Then, to identify the liquefaction hazard level in the dam foundation, the Liquefaction Potential Index values were grouped and cumulative distributions of I_L were established.

Fig. 18 illustrates the distribution of the calculated I_L values of 20 CPTs sounding using the Robertson method. The results show that only 4% of the untreated points have an I_L less than 5 and 91% of the treated points have an I_L greater than 15. So, according to Iwasaki classification criteria, the liquefaction failure is extremely high in the site of Sidi El Barrak dam. However, after vibrocompaction, it can be observed that 91% of the compacted points have an I_L smaller than 5 which indicate that the liquefaction risk is low.

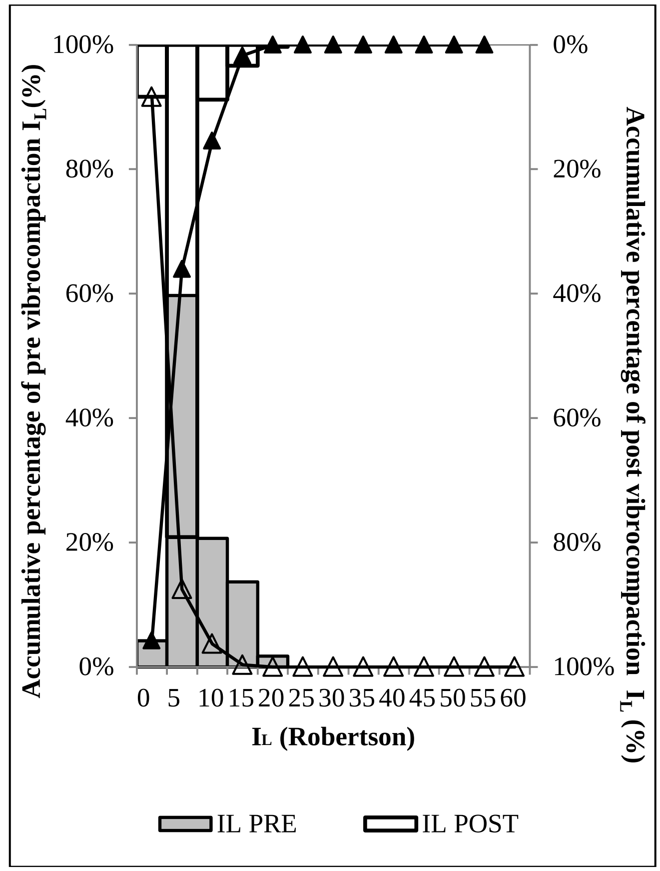

Fig. 18. Distribution of Calculated I_LValues obtained from Robertson method

CONCLUSION

The detailed geotechnical investigation including SPT and CPT tests were used effectively to identify the liquefaction potential of the foundation of Sidi El Barrak dam. Based on this liquefaction analysis, the following conclusions are reached:

The liquefaction evaluation results based on the SPT data show more similarity to those based on the CPT data. Indeed, this case study demonstrates the successful mitigation of the liquefaction risk under the design earthquake. The factor of safety against liquefaction is obtained from SPT and CPT based simplified procedures. The results show that the undensified alluvial sands of foundation were prone to liquefaction hazard (FS < 1). However, after vibrocompaction, the dam foundation was not susceptible to liquefaction (FS > 1).

Before soil densification, the liquefaction evaluation using CPT-data shows probabilities over 35 % which mean that the foundation is exposed to the liquefaction phenomenon. After vibrocompaction, the site presents low probability of liquefaction;
The calculated Liquefaction Potential Index suggests for the untreated soils highest frequency occurring at highest I_L class. For the treated layers, the percentage of liquefaction failure in the high risk class is negligible or absent.

REFERENCES

El Ouni M.R., Bouassida M., Braja M.D. [2009]. Vibrocompaction improvement of Tunisian liquefiable sands, Proceedings of the 17th international conference on soil Mechanics and Geotechnical Engineering, pp.1-4.

Holzer T., Toprak S., Bennett M.[2003]. Application of the liquefaction potential index to liquefaction hazard mapping. Procceding from the 8th US - Japan workshop on Earthquake resistant design of Lifeline facilities and counterness against liquefaction, Tokyo, Japan.

Juang C.H., Andrus R., Chen J.[2000]. Risk- based liquefaction potential evaluation using Standard Penetration Test. Canadian Geotechnical Journal, No 37,pp. 1195-1208.

Juang C.H., Chen C.J. [2000]. A rational method for development of limit state for liquefaction evaluation based on shear wave velocity. International Journal for numerical and analytical methods in geomechanics, No 24,pp.1-24.

Juang C.H., Jiang T.[2000]. Assessing probabilistic methods for liquefaction potential evaluation. Soil dynamics and liquefaction, ASCE geotechnical spatial publication, pp48-62.

Juang C.H., Yuan H., Lee D.H., Ku C.H.[2002]. Assessing probability-based methods for liquefaction evaluation. Journal of Geotechnical and Geoenvironmental Engineering, ASCE,pp. 241-258.

Juang H., Suber J.[2000]. Estimation of liquefaction - induced horizontal displacement using artificial neural networks. Canadian Geotechnical Journal, No 38,pp 200-207.

Lee H.D.,Ku C.S.,Yuan H.[2003]. A study of the liquefaction risk potential at Yualin, Taiwan. Engineering geology Journal, pp.97-117.

Robertson P.K., Campanella R.G.[1985]. Liquefaction potential of sands using the CPT. Journal of Geotechnical engineering,pp. 298- 307.

Robertson P.K., Wride C.[1998]. Evaluating cyclic liquefaction potential using the cone penetration test. Canadian Geotechnical Journal, pp.442-459.

Seed B.[1979]. Soil liquefaction and cyclic mobility evaluation for level ground during earthquake. Journal of Geotechnical Engineering, pp.201-255.

Seed B., Idriss I.M. [1971]. Simplified procedures for evaluating soil liquefaction potential. PROC.JSME, ASCE, Vol 97,SM9,pp. 1249-1273.

Seed B., Idriss I.M., Arango.I.[1983]. Evaluation of liquefaction potential using field performance data. Journal Geotech.Eng.ASCE, Vol., 109, No 3,pp 458-482

Seed H.B., Tokimatsu K., Harder L.F.,Chung R.M.[1985]. The influence of SPT Procedures in soil liquefaction resistance evaluations. Journal of geotechnical Engineering, ASCE, Vol., 111, No 12,pp. 1425-1445.

Technical document.[1990]. Drainage and treatment of the foundation of Sidi El Barrak dam (in French). Ministry of agriculture and Hydraulic resources of Tunisia.

Trifunac MD., Brady, AG. [1975]. A study on the duration of strong earthquake ground motion. Bulletin of the Seismological Society of America, pp.581-626.

Youd T.L., Andrus M., Idriss M. [1997]. Proceedings of the NCEER workshop on evaluation of liquefaction resistance of soils, pp. 1-40.

Youd T.L.,Idriss I.M., Andrus R.D., Marcusson F., Robertson P.K., Seed R.B., Stokoe K.H.[2001]. Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. Journal of Geotechnical and Geoenvironmental engineering,pp. 817-833.