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Validity of Juang's Method to Predict the Liquefaction Potential of the Soils of Sablette Algiers

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Abstract. This work presents a deterministic and probabilistic analysis of the liquefaction potential of soils. Soil liquefaction is a process leading to the total loss of shear strength of loose sands under seismic stresses by increasing pore pressure. It is accompanied by deformations.

Certain factors related to the characteristics of the ground and the seismic excitation can significantly influence the appearance or not of the phenomenon.

Empirical methods of liquefaction potential assessment are widely used. The probabilistic method has been widely used for estimating the probability of liquefaction. Initially, the uncertain parameters are modeled by normal random variables. A parametric study of the coefficients of variation of the random variables showed that the number of strokes $(N1)_{60}$ of the SPT test is the parameter that has the greatest influence on the probability of liquefaction.

This study focuses on the liquefaction potential by methods based on the results of the in-situ "Standard Penetration Test"

Through the results obtained in this study of the site of Sablettes in Algiers, in general, there is a risk of liquefaction of the soil in the sandy layers, and we can say that the site studied has an average liquefaction potential. According to the classification of liquefaction potential proposed by Juang et al., (2012).

Keywords. Liquefaction, Interstitial Pressure, Liquefaction Potential, SPT Tests.

INTRODUCTION

One of the spectacular aspects of the seismic event is the collapse of the structures, following the degradation of the shear resistance of pulverulent and loam soils of low compactness.

The term liquefaction is used to describe this process of loss of resistance of such soils, with as a consequence the manifestation of superficial ruptures, the collapse or the collapse of the surface, the formation of craters and the rise of the sand, loss of load-bearing capacity, as well as landslide.

Certain soils composed of fine grains (sand and silts) whose particle size is in a certain critical zone are susceptible when they are bathed in the water table and are in an initially insufficiently tight state, giving rise to liquefaction phenomena.

Liquefaction corresponds to a process of loss of shear resistance during which an accumulation of interstitial pressures manifests itself through the cycles of seismic stress, to the detriment of a fall in the effective stresses.

The shear resistance, because it is proportional to the effective stress according to the Mohr-Coulomb fracture criterion, decreases and cancels out after a certain number of cycles, thus initiating the behavior of a liquid material not resistant to shear (Bouafia, 2010).

A process in which, under the effect of several cycles of alternating shear deformations of great amplitude, the pressure of the water included in the interstices of the grains rises from cycle to cycle until it equals the pressure of gripping the medium, losing all or part of its shear resistance, behaves like a fluid and becomes unable to withstand the vertical loads brought by the above structures or formations.

The works are sometimes subjected to considerable subsidies and dumping. Liquefaction is a phenomenon in which the shear resistance of soil is reduced by the shaking of an earthquake or any other fast loading.

Liquefaction occurs in saturated powdery soils, that is, soils where the space between the different particles is filled with water. This water exerts an interstitial pressure on the soil particles, resulting in a decrease in the effective stresses in the soil and thus a decrease in its shear resistance that, in the final state, can become zero.

Before an earthquake, the water pressure is relatively low. However, the tremors of an earthquake can cause a considerable increase in the pore pressure of the water to the point where soil particles can easily move concerning each other. According to the RPA 99 corrected in 2003, the safety factor is 1.25.

DETERMINISTIC STUDY OF SOIL DEPLETION POTENTIAL (SABLETTES SITE)

Site Status

The zone to be studied is located inside the Park des Sablettes. The base of the project is delimited: - To the North, by the Mediterranean Sea; - To the South and to the East, by a complement of bare ground, - To the West, by a complement of ground and the Ferris wheel (Fig. 1).

Fig.1. Project Status Plan (Google earth).

Regional geology

According to the geological map of Algiers N°21 at 1/50.000th the region of Central Algiers is located in the western part of the Mitidja. It is represented from the geological point of view by the following recent and old Pliocene formations (Fig. 2): - The metamorphic dome is represented by mica schists, gneiss, and limestone's that surface at Bouzaréah. - A stratigraphic deficiency of secondary deposits. - The tertiary unconformably covers the metamorphic substratum; it is represented by: the lower Miocene formed of sandstone and powdingues, the upper Miocene is represented by the conglomerate and sandstone, and by the lower Pliocene, represented by the argilo deposits—marls surmounting the middle Pliocene represented by the Astien of sandstone-limestone nature (Yassini, 1973). - The quaternary and the current are defined by the consolidated dunes of the Pleistocene, the red sands, the recent alluvium, and the current dunes.

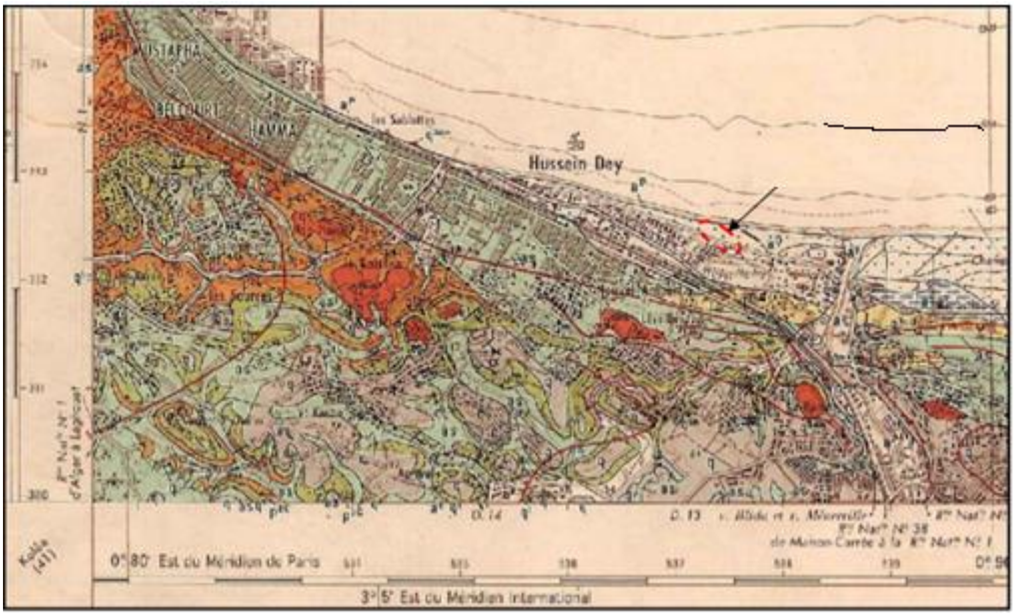

Fig.2. Extract from map N°21 Algiers bis at 1/50.000th.

Seismic Context

The wilaya of Algiers is located in the high seismicity zone. For the dynamic calculation of the work, it is necessary to refer to the Algerian seismic regulation in force (RPA 99 version 2003) (Fig. 3).

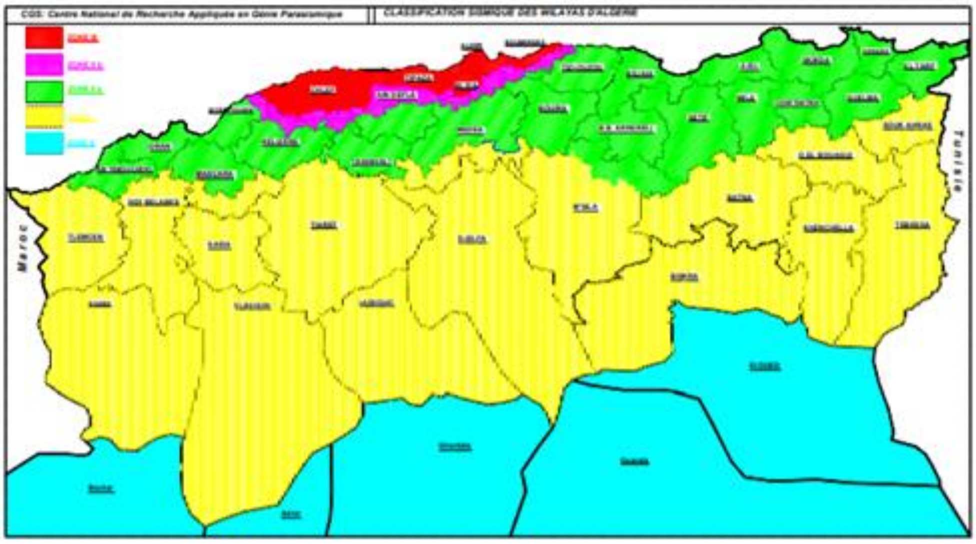

Fig.3. Seismic zoning map of the national territory (based on CGS 2003).

The Algiers region belongs according to the seismic zoning map (CGS, 2003) to Zone III with high seismicity.

Given the sandy nature of the soil in place (10<N<50), the soil is classified as category S3, corresponding to loose soil, and this is according to table 3.2 RPA 99/2003 site classification.

RESULTS OF THE GEOTECHNICAL INVESTIGATION PROGRAM

The program of the geotechnical reconnaissance campaign consisted of the execution of:

-Three (03) boreholes of 15.00 meters in depth, equipped with piezometers (Table 1). -One (01) pressure borehole 15.00 meters deep with tests every 1.50 meters (Table 1). -Five (05) dynamic penetrometer tests distributed over the accessible site assembly pushed to refusal rated P1-P2-P3 – P4 and P5 (Table 1). - SPT sand test.

The results of the penetrometer tests are represented by penetrograms that show the variation of the peak resistance (Rp) as a function of depth (Table 2).

The penetrations noted P-01-P-02-P-03-P-04 and P-05 correlated with the cored boreholes SC01-SC02 and SC03 determining that all the points recorded false refusals at the port rip-rap blocks.

The value of the dynamic resistance in peak Rp is much more than 100 bars from the surface up to 1.20 m depth where it is marked the need for refusal of the peneometric test. This soil layer with RP> 100 bar corresponds to the rock fill layer of the port area (Table 2).

For the calculation of the exact number of strokes measured by the S.P.T test (Table 3), the raw values obtained N1, N2 and N3 of the S.P.T test carried out and only the N2 and N3 values will be taken into account, because the value N1 corresponds to the modified part of the soil and therefore to be eliminated, so the value of N that we will take into consideration will be: $N = N2 + N3$ (1).

Table 1. The UTM coordinates of the surveys carried out.

N°	SC 01	SC 02	SC 03	PS 01	P-01	P-02	P-03	P-04	P-05
X(m)	510 855	510 807	510 834	510 832	510 863	510 792	510 791	510 862	510 829
Y(m)	4066 395	4006 399	4 066 365	4 066 379	4 066 404	4 066 410	4 066 355	4 066 355	4 066 383

Table 2. The results of the penetrometer tests at the variation of the peak resistance (Rp) as a function of depth.

N°	P-1	P-2	P-3	P-4	LP5
Depth (m)	00.20-01.20	00.20-00.60	00.20-00.60	00.20-00.40	00.20-00.80
Peak resistance (bars)	Rp>100	Rp>100	Rp>100	Rp>100	Rp>100

Table 3. The raw values of the S.P.T tests (N1, N2 and N3).

N°	Depths (m)	Number of strokes			N_2+N_3	Soil classification according to XP P94-011
		N_1	N_2	N_3		
	09.55-10.00	3	5	7	15.1	
SC 01	11.05-11.50	3	5	8	13	
	12.55-13.00	2	5	7	12	
	14.55-15.00	4	8	13	21	
	09.55-10.00	2	5	9	14	
SC 02	11.05-11.50	4	7	12	19	Moderately dense soil
	12.55-13.00	3	7	13	20	
	14.55-15.00	5	9	16	25	
	09.55-10.00	2	5	9	14	
SC 03	11.05-11.50	5	9	15	24	
	12.55-13.00	4	8	16	24	
	14.55-15.00	5	9	18	27	

DETERMINISTIC STUDY OF SITE LIQUEFACTION POTENTIAL

In the deterministic liquefaction study, the safety factor:

$Fs = CRR/CSR\ (2)$

The safety factor was calculated for each depth. The soil is supposed to be liquefiable if the safety factor Fs<1, and it is not liquefiable if Fs>1.

We note from the results that the safety factor gives liquefiable layers. And based on the results obtained we have a risk in the following depths: The findings of the cases of rupture of foundation soils by liquefaction showed that the risk of liquefaction is present throughout the depth of the borehole except the SC03 boreholes at depths of 13m. 55 to 14.00 meters and 14m. 55 to 15.00 meters.

To eliminate or reduce the risk of liquefaction, one or more of the following measures should be implemented:-A permanent drawdown of the water table level. - Densification of liquefiable layers (preloading, dynamic compaction, etc.). -An improvement in the permeability of liquefiable layers by making drains of coarse materials. - Substitution of suitably compacted material for liquefiable layers.

Correction of SPT values

Example calculation: (SC01- 1st point).

$N1 = 3, N2 = 5, N3 = 7 N = N2 + N3 = 5 + 7 = 12.$

$P'0 = (\delta h - 1) x\ h$ (3) = $P'0 = (1.80 - 1)\ x\ 10 = 8.00\ t\ /m^2$

So for depth correction:

$N' = N + (\frac{35}{P'0} + 7))$ (4) = $12 + (\frac{35}{8} + 7)), N' = 14.33$

For Water Table Correction:

N’’ = 0.50, N’ + 7.50 , N’’ = 0.50 x 14 + 7.50 , N’’ = 14.50, N’’ = 14.50

N’: is the value taken into consideration for estimating the compactness of the sand.

The analysis of the S.P.T results shows that we are in the presence of medium dense to dense sand, sometimes very dense in depth.

Given the presence of water and the nature of the fine soil, a verification of liquefaction is essential.

Table 4. Correction of SPT Values.

N°	Depth (m)	Number of strokes			Resistance to penetration			Friction angle*	Friction angle**	Compactness
		N1	N2	N3	N	N'	N''			
SC 01	09.55-10.00	3	5	7	12	14	14.5	30-36°	35-40°	Average
	11.05-11.50	3	5	8	13	15	14	30-36°	35-40°	
	12.55-13.00	2	5	7	12	14	14.5	30-36°	35-40°	
	14.55-15.00	4	8	13	21	23	19	30-36°	35-40°	
SC 02	09.55-10.00	2	5	9	14	16	15.5	30-36°	35-40°	
	11.05-11.50	4	7	12	19	21	18	30-36°	35-40°	
	12.55-13.00	3	7	13	20	22	18.5	30-36°	35-40°	
	14.55-15.00	5	9	16	25	27	21	30-36°	35-40°	

*Friction angle according to Terzaghi.

**Friction angle according to Meyghorf.

CSR Cyclic Stress Calculation (Survey #01)

Among the different methods of assessing liquefaction potential, the most commonly used is the one developed by Seed and Idriss in 1971 in accordance with discussions at the NCEER Workshops in 1996 and NCEER/NSF in 1998 (Youd and Idriss, 2001).

This method consists of determining the cyclic stress ratio CSR (Cyclic Stresse Ratio) and the cyclic resistance ratio CRR (Cyclic Resistance Ratio), determined from in-place tests S.P.T Standard Penetration Test.
The comparison of these two rates allows the definition of the safety factor with regard to the risk of liquefaction:
$FSL = CRR / CSR$ (5)
Thus, if the shear stress induced by the CSR earthquake:
$CSR = \tau_{\alpha\varpi} / \sigma' v$ (6)
The CSR earthquake is lower than the cyclic shear stress $CRR_{7,5}$ there is no risk of liquefaction. Similarly, a safety factor is determined from the ratio of the cyclic shear stress $CRR_{7,5}$ (7.5 magnitude reference earthquake) to the earthquake-induced shear stress multiplied by the magnitude scale factor.
The different steps for the calculation are the determination of:
1- The effective stress in the middle of the sand layer;
2- The total stress in the middle of the sand layer;
3- Stress reduction factor rd (according to Liao and Whitman, 1986).

- $rd = 1 - 0{,}00765\ h$ (7) pour $h \leq 9{,}15$ m
- $rd = 1.174 - 0{,}0267\ h$ (8) pour $9{,}15 \text{ m} \leq h \leq 23$ m
- $rd = 0.744 - 0{,}008\ h$ (9) pour $23 \text{ m} \leq h \leq 30$ m

h: the depth considered.
4- The shear stress induced by the CSR earthquake:
$CSR = \tau_{\alpha\varpi} / \sigma'_{\varpi 0} = 0{,}65\ (amax / g)\ (\sigma_0 / \sigma'_0)\ rd$ (10)
With: amax: maximum surface acceleration or zone acceleration coefficient. g: gravity acceleration. rd: coefficient reflecting soil flexibility. γ : Soil-specific weight (t/m^3). h: depth (m)
5- Correction of the SPT test values factors:
$(N1)_{60} = N\text{m}\ C\text{N}\ C\text{E}\ C\text{B}\ C\text{R}\ C\text{S}$ (11)
With: Nm: the value of N measured with: .
$N_m = N1 + N2$
C_N: land weight correction factor :
$C_N = (P_a / \sigma'_0)^{0.5}$ (12)
P_a : atmospheric pressure in the same unit as σ'_0,σ'_0: the effective vertical stress at the depth considered. C_E C_B C_R C_S: correction factors whose value is unit = 1.
When the $(N1)_{60}$ values are greater than 30, there is no risk of liquefaction and the safety factor is high.
6- The correction of the percentage of fines where it is greater than 5 (the percentage of fines being variable).
7- $(N1)_{60CS} = \alpha + \beta\ (N1)_{60}$ (13)
With :
$\alpha = \exp\ [1.76 - \{[190 / (\text{percentage of the fines})^2]\}$ (14a)
$\beta = [0.99 + [(\text{percentage of the fines})^{1.5} / 1000]$ (14b)

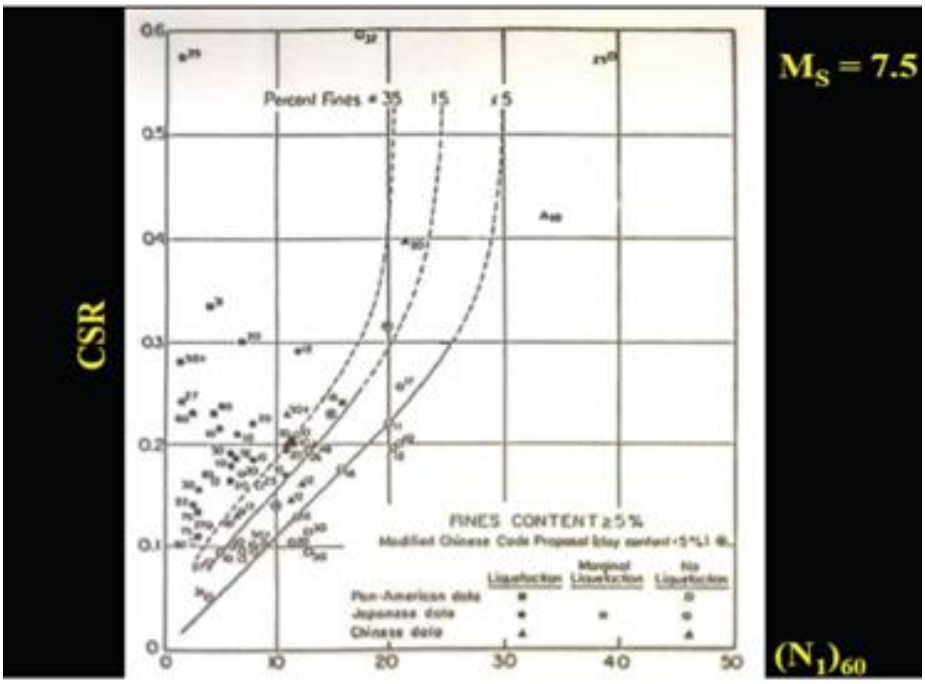

Fig.4. Liquefaction risk based on CSR and $(N1)_{60}$.

8- Cyclic shear stress $CRR_{7,5}$ given according to $(N1)_{60CS}$

$$CRR_{7.5} = \left(\frac{1}{34-(N1)_{60cs}} + \left(\frac{(N1)_{60cs}}{135}\right) + \left(\frac{50}{[10(N1)_{60cs}+45]^2}\right) - \left(\frac{1}{200}\right) \right) \quad (15)$$

9- The scale factor of the MSF magnitude for a given magnitude:

$MSF = 10^{\,2.24} / M^{\,2.56}$ (16)

10- FS safety factor with FS less than 1 indicating soil liquefaction :

$FS = (CRR_{\,7.5} / CSR)\ x\ MSF$ (17)

Laboratory test results

The cored survey we conducted allowed us to retrieve redesigned samples, resulting from the SPT test, sent to the laboratory for physical identification testing.

The particle size analysis was used to classify soils as grainy (Table 5).

According to NF P18-011, the results show that the soil testing is not aggressive (Table 6).

Table 5. Particle size analysis result.

N°	Depth (m)	<2 mm	<4μ	<80μ
	09.55-10.00	99.47	91.91	3.49
SC 01	11.05-11.50	99.95	93.09	1.99
SC 02	09.55-10.00	99.41	95.18	7.26
	11.05-11.50	99.69	93.24	10.91
SC 03	12.55-13.00	99.84	93.64	4.96
	14.55-15.00	99.71	92.92	5.18

Table 6. Results of chemical analyses.

N°	Depth (m)	CO_3 (%)	Carbon dioxide (%)	Chloride (%)	Sulphate (%)
SC 01	09.55-10.00	34.49	15.18	0.29	Traces
SC 02	11.05-11.50	37.07	16.31	0.41	Traces
SC 03	12.55-13.00	36.21	15.93	0.35	Traces

Verification of liquefaction risk

Vibrations (earthquakes) in some saturated sandy soils because of alternating deformations and increases in pore pressure, which induce a momentary loss of shear resistance.

Soils susceptible to liquefaction are generally clean or silty sands located in the first twenty (20) meters of depth, saturated with water and with a relatively uniform particle size corresponding to a Cu uniformity coefficient of less than 15.

($Cu = D_{60}/D_{10} < 15$) and a diameter of 50% (D_{50}) between 0.05 and 1.50mm; D_{60}, D_{10}, and D50 represent the sieve diameters corresponding to the 60%,10%, and 50% passageways respectively of the soil samples considered (Table 6).

The liquefaction of soils during an earthquake is a process of decreasing their shear resistance; it is most often observed in saturated sandy deposits.

Table 7. Analysis of liquefaction susceptibility by particle size.

N°	SC01				SC02				SC03			
	D10	D50	D60	Cu	D10	D50	D60	Cu	D10	D50	D60	Cu
09.55-10.00	0.11	0.26	0.33	3	0.10	0.28	0.32	3.2				
11.05-11.50	0.14	0.28	0.32	2.28	0.008	0.25	0.32	40				
12.55-13.00									0.94	0.25	0.30	0.32
14.55-15.00									0.94	0.27	0.32	0.34

The layers liable to be liquefied are those whose coefficient of uniformity Cu is less than 15.00 and D50 between 0.05 and 1.50 mm, i.e.:

- For borehole SC01: - from 09.55 to 10.00 m depth. - from 11 to 11.50 m in depth.
- For borehole SC02: - from 09.55 to 10.00 m depth.
- For borehole SC03: - T 12.55 to 13.00 m depth. - 14.55 to 15.00 m depth.

STANDARD PENETRATION TEST (SPT) LIQUEFACTION POTENTIAL ASSESSMENT

We will consider the SPT results obtained at the SC01 borehole depth of -09.55 m - 10.00 m and the values are: N1 = 3; N2 = 5 and N3 = 7.

Where : $N_m = N2 + N3 = 12$

The calculation will be carried out for the sand layer whose density is $\gamma = 1.8\ t/m^3$ surmounted by a fill of 08.00 m thickness of density $\gamma = 2.00\ t/m^3$.

1 - The effective stress: $\sigma'_{v0} = (2-1)\ x\ 8 + (2/2)\ x\ (1.8-0.8) = 8.80\ t/m^2$

2 - Total stress: $\sigma_v = 2\ x\ 8 + (2/2)\ x\ 1.80 = 17.80\ t/m^2$

3 - The reducing coefficient: $rd = 1.174 - 0.00267\, h = 1.1473$

4 - The shear stress induced by the CSR earthquake

$CSR = \tau_{av} / \sigma'_{v0} = 0.65\ (amax\ /\ g)\ (\sigma 0\ /\ \sigma' 0)$

With: $amax = A\ x\ g\ (sand\ coefficient) = 0.40\ x\ 1.50\ x\ g = 0.60\ g$

Zone III A = 0.40 according to R.P.A 99 version 2003

A = 0.40 which corresponds to the acceleration of the site in zone III for a group 1 A; of use 2, according to R.P.A 99 version 2003.

5 – Correction of SPT test value factors:

$(N1)_{60} = 12\ x\ (8.8\ /\ 17.80)\ 0.5 = 12\ x\ 0.695 = 8.44$

6 - Correction of the percentage of fines where it is greater than 5 (the percentage of fines may be variable)

$$\alpha = exp\ [1.76 - \{[190\ /\ (7.26)^2]\} = 0.16$$

$$\beta = [0.99 + [(7.26)1.5\ /\ 1000] = 1.009$$

if the percentage is less than 5%,

$(N1)_{60CS} = 0.16 + 1.009\ x\ 8.35 = 8.58$

Cyclic shear stress CRR7.5 given as a function of $(N1)_{60CS}$

$$CRR_{7.5} = \left(\frac{1}{34 - (N1)_{60cs}} + \left(\frac{(N1)_{60cs}}{135} \right) + \left(\frac{50}{[10(N1)_{60cs} + 45]^2} \right) - \left(\frac{1}{200} \right) \right)$$

$$CRR_{7.5} = \left(\frac{1}{34 - 8.58} + \left(\frac{8.58}{135} \right) + \left(\frac{50}{[10 \times 8.58 + 45]^2} \right) - \left(\frac{1}{200} \right) \right)$$

$CRR_{7.5} = 0.0999$

7 - The scale factor of the MSF magnitude for a given magnitude : $MSF = 0.999$

8 - The safety factor FS with FS less than 1.25 indicating the liquefaction of the soil :

FS = 0.11 ⇒FS <1.25 liquefiable soil

The calculation for the other points is done in the same way and we find the results in the tables below:

Table 8. Tablecloth correction N" Soil classification according to XP P94-011.

N°	Depth (m)	N2 + N3	Correction N''	Classification according to norme XP P94-011
SC 01	09.55-10.00	12	14.5	Medium dense soil
	11.05-11.50	13	15	
SC 02	09.55-10.00	14	15.5	
	11.05-11.50	19	18	
SC 03	12.55-13.00	24	20.5	
	14.55-15.00	27	22.00	

Table 9. Calculation of the liquefaction potential assessment by the SPT method.

N°	Depth (m)	CSR	$(N1)_{60}$	(%)*	$(N1)_{60CS}$	CRR7,5	MSF	FS	Liquefaction risk
SC 01	09.55-10.00	0.905	8.44	7.26				0.11	liquefiable soil
	11.05-11.50	0.928	18.55	10.91				0.23	liquefiable soil
SC 02	09.55-10.00	0.989	9.8	3.49	9.8	0.110	0.999	0.124	liquefiable soil
	11.05-11.50	0.908	13.3	1.99	13.3	0.1426	0.999	0.156	liquefiable soil
SC 03	12.55-13.00	0.889	34.32	4.96	**34.32**	**$(N1)_{60}$ >30 non liquefiable soil**			
	14.55-15.00	0.941	40.23	5.18	**40.23**				

*Percentage of fines below 74 µ .

ESTIMATING THE LIKELIHOOD OF LIQUEFACTION

The probability of liquefaction is estimated by the relationship of Juang et al., (2002):

$$P_L = \frac{1}{1+\left(\frac{FS}{1.05}\right)^{3.8}} \quad (18)$$

The results of the liquefaction potential calculation are presented in table 10. The probability of liquefaction is in the order of 99% with an FS factor < 1.

According to Table 3, the soil studied is certain to liquefy.

Table 10. Calculation of the liquefaction potential estimate.

N°	Z(m)	P_L
SC 01	09.55-10.00	0.9998109
	11.05-11.50	0.9968905
SC 02	09.55-10.00	0.9997019
	11.05-11.50	0.9992871

Figure 5 shows the change in liquefaction probability as a function of the MSDS safety factor for the given site using the Juang et al., (2002) method.

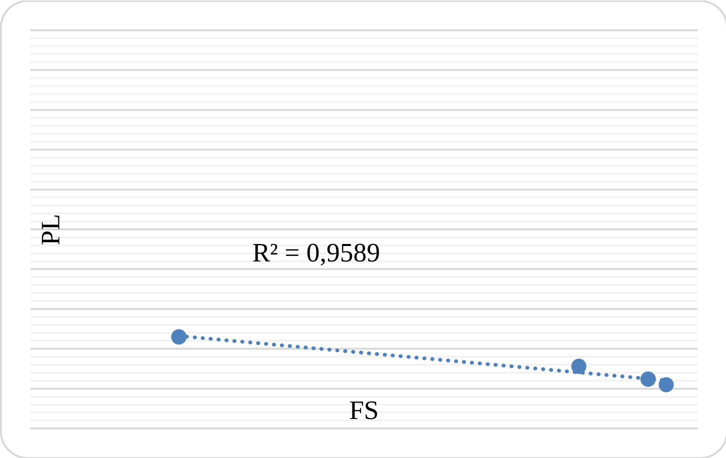

Fig.5. Variation of PL according to FS.

The overall liquefaction index, IL, is used to assess the impact of the depth and thickness of liquefiable horizons for a soil column (Iwasaki et al., 1982):

$IL = \int_0^{20} FL\ w(z)\ dz$ (19)

Where: - FL =0 If materials are not qualitatively susceptible. - FL = 1- FS Si FS< 1
FL = 0 Si FS ≥ 1 -w(z): weight function for depth given by:

$w(z) = 10.0 - 0.5\ Z$ (20)

Z: depth (m)

where:

$IL = \int_0^{20}(10 - 0.5\ Z)FLdz = \sum_{i=0}^{N}(10 - 0.5\ Z)FL\ (h_{i+1} - h_i)$ (Luna, 1995, Luna and Frost, 1998).

The value of IL varies from 0 for a non liquefiable site to 100 for a very strongly liquefiable site. Several classes are distinguished: no liquefaction: non liquefiable areas or IL = 0; unlikely liquefaction: 0 < IL <5; likely liquefaction: 5 < IL < 15; almost certain liquefaction: IL > 15.

Table 11. Calculation of the value of IL.

N°	Z(m)	FS	FL	IL
SC 01	09.55-10.00	0.9998109	0.0001891	0.000442548
	11.05-11.50	0.9968905	0.00310952	0.00594696
SC 02	09.55-10.00	0.9997019	0.00029809	0.0006707
	11.05-11.50	0.9992871	0.00071293	0.00136348

Based on the value of IL: IL=0.0021, liquefaction is unlikely.

CONCLUSION

The liquefaction of a site is a particular aspect of loose granular soils and can cause serious damage to buildings, so it is important to be able to predict the behavior of soils in the face of the liquefaction problem.

The assessment of liquefaction potential by empirical and semi-empirical methods has become increasingly popular and usable. Engineers are increasingly using conventional methods based on in situ tests such as the SPT test. These methods use deterministic relationships, for the development of boundary curves to observe the appearance or not of liquefaction. These methods are based on the calculation of a safety factor (ratio of CRR to CSR) Soil saturation conditions are generally assumed. Some limitations impose uncertainties on their use. For this, we use the probabilistic method to see the uncertainties influencing the liquefaction, thus introducing them into the calculations of the liquefaction probability.

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