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EFFECTS OF AN EARTHQUAKE ON A LIQUEFACTION OF A HARBOUR TERMINAL

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ABSTRACT

The paper presents disorders induced by an earthquake of magnitude 7 on the Richter scale on a harbour terminal in urban site in Algeria. Geotechnical investigation post-event highlighted disorders in the underground made up of a hydraulic fill. Investigations included laboratory identification, laboratory mechanical tests and in situ tests. The diagnostic carried out on surface installations and the analysis of the geotechnical tests results let think of an under-ground liquefaction in the 6-9m depth zone. This is confirmed using predictive methods based primarily on interpretation of the CPT and SPT in-situ tests. Results within present site soil state show serious probability of liquefaction of the ground in the event of future earthquake of magnitude higher than 6. Need for seeking solutions of liquefiable site stabilization and reinforcement is thus highlighted.

Keywords: Earthquake, urban site, liquefaction, CPT, SPT, stabilization

INTRODUCTION

Evaluation of cyclic liquefaction potential is a major concern for large geotechnical research group as well as civil engineers designers especially those concerned by maritime works. Indeed, number of reclamation zones, embankment and platforms are gained onshore by fulfilling with cohesionless soils. Comprehensive standard penetration test (SPT) method has been developed to estimate the potential liquefaction due to earthquake loadings by H. B. Seed and his colleagues [Seed, (1979), Seed and Idriss (1971), Seed et al. (1969)]. More recently, because of its greater repeatability and continuity of its profile cone penetration test CPT method became even popular these last decades. Robertson and Wride (1998), Robertson and Campanella (1985) described soil liquefaction phenomenon, reviewed suitable definition and provided updated methods to evaluate cyclic liquefaction using these field data. Different state-of-art papers exist, as Robertson et al. (1992), Juang et al. (2003), describing in detail reflection and analysis of soil liquefaction.

The present paper deals with a comparison of a potential cyclic liquefaction evaluation due to the earthquake loading, using present site soil state loaded by 2003 Algeria earthquake. Calculation using both SPT and CPT soundings data give interesting results and rapid analysis of cyclic liquefaction by grading curves is also included

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DAMMAGE ON CONTAINER TERMINAL RECLAMATION PLATFORM

A disastrous earthquake taken place at Boumerdes (60km East of Algiers, Algeria) in May 2003 within a magnitude 6.8 (Richter scale), caused huge human and material damage. Its important scale creates significant disorders of the ground, basements and works in a large zone of center of Algeria. The container terminal platform of the port of Algiers (320m x 460m and 9.20 to 11.0m depth, figure 1), consists of an old embankment (Mole El Hadjar, figure 2a) and a new one hydraulic fulfilled ten years ago (dock N°7).

Geotechnical survey of the site was carried out after the event. It included 5 borings (15m depth) as well as 31 SPT tests (Standard Penetration Test) and 42 CPT (Cone Penetration Test) distributed on the whole container terminal platform (figure 2b). The land survey carried out shows 15 cm to 20 cm settlement on the embankment. The major disorders are observed on the mole of El-Hadjar, with 4 important cavities parallel to the berth n° 30 (Figure 2c). Cracks on the revetment are also observed on the boundary zone between the ancient and new part of the embankment, due to presence of massive structures of the old quay contiguous to a flexible band of hydraulic embankment. Cavities were noted in the median zone of the wet dock (Darse N°7, figure 2a) and under a great part of the crane rails.

CYCLIC LIQUEFACTION POTENTIAL ANALYZIS

As it is well known, the risk of liquefaction of coarse-grained soils increases in the case of earthquake occurrence. Undrained shear cyclic stress τ_{rc} decreases, if the conjugation of the three main factors occurs: strong cyclic loading, weak soil compactness, low permeability ratio.

Tentative of application of different existing tools developed worldwide is made in order to verify the present state of Algiers container terminal case vis-à-vis potential liquefaction using parameters of future earthquake event.

Having number combined boreholes, SPT and CPT field data, another aim of the present paper is a comparison of results of different methods exposed here below.

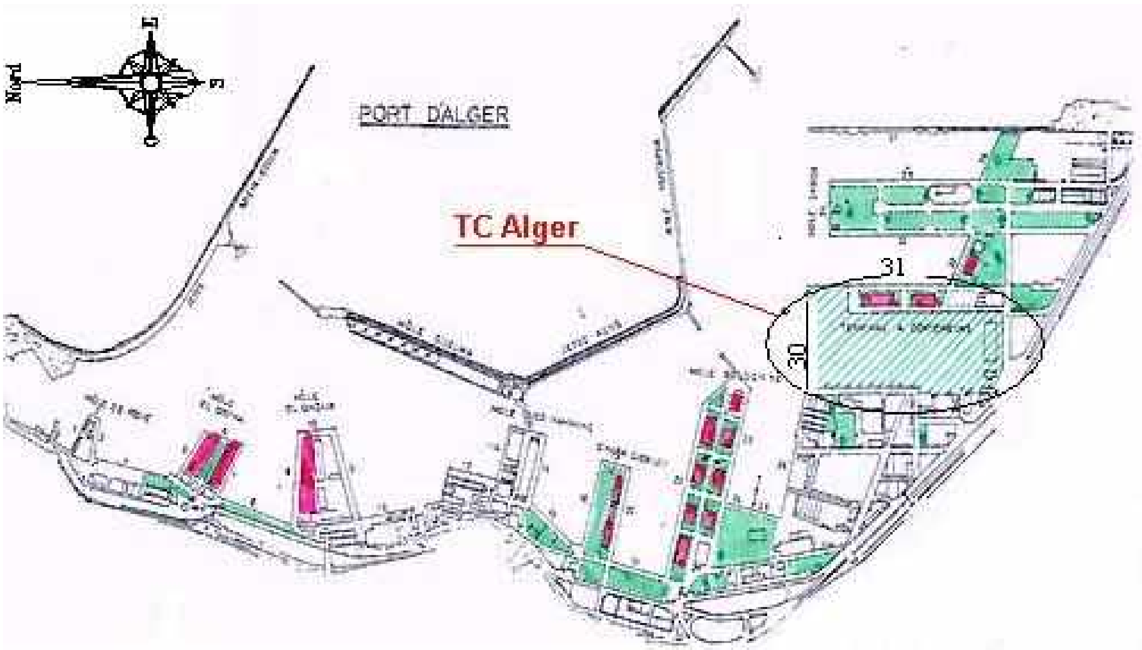


Figure 1. Container terminal at Algiers port

Cyclic liquefaction analysis using grading curves

Thompson & Emery (1976) proposed granulometric envelopes of liquefied sand during earthquakes in Japan and Lee & Fitton (1968) laboratory test data of liquefiable sands. If the grading curves of the 5 Algiers boreholes characteristics (S1 to S5, figure 2b), are put on the same figure, one can notice that the whole grading curves are concentrated inside the two envelopes. Material of Algiers container terminal embankment could be thus included in the family of potentially liquefiable sands.

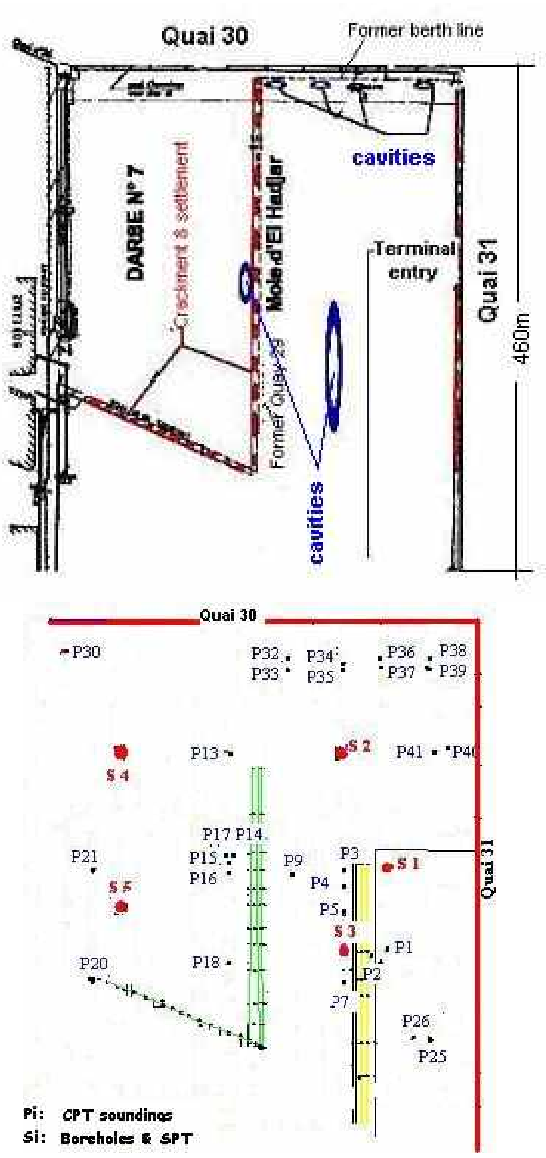


Figure 2. (a) Container terminal damage localisation ; (b) In situ tests localisation

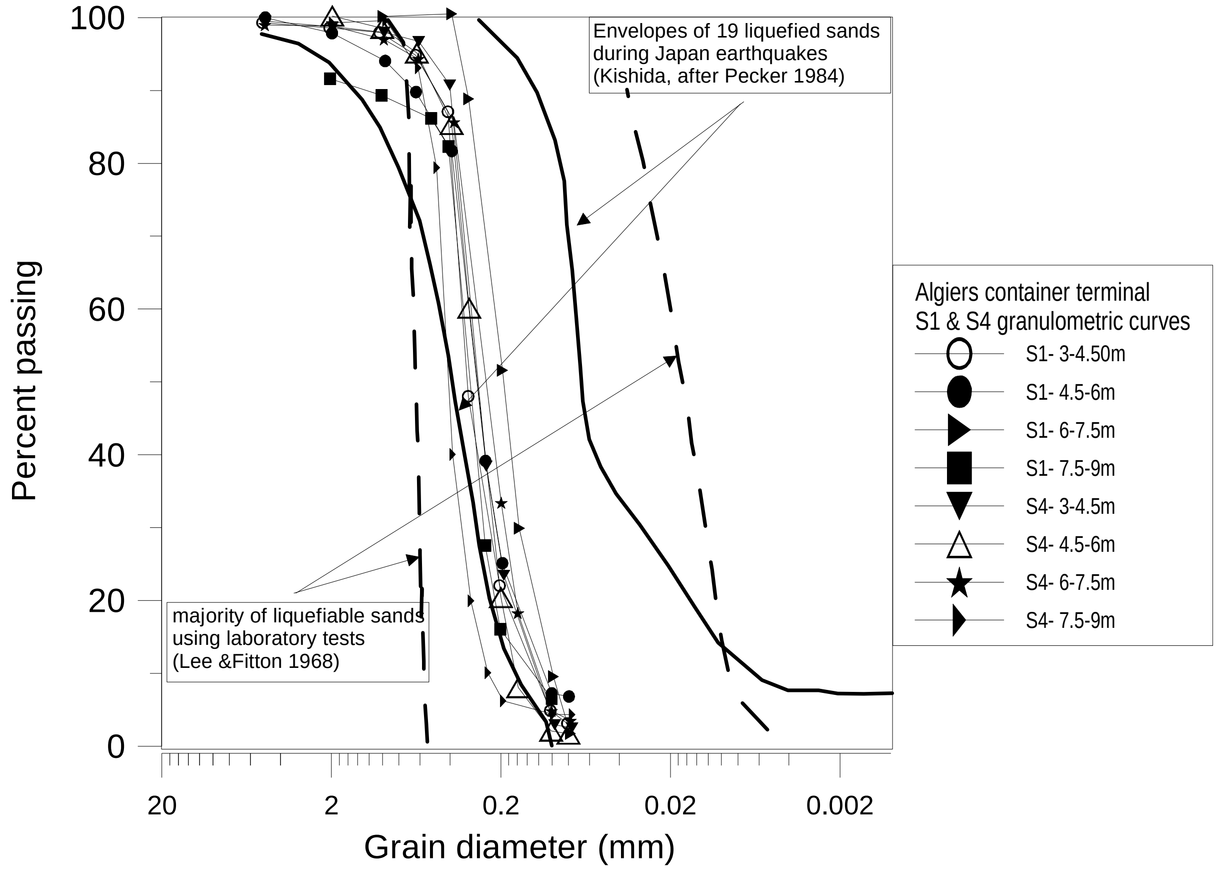


Figure 3. Algiers S1 & S4 grading curves within granulometric envelopes of liquefied sand

Cyclic potential liquefaction assessment using SPT tests

SPT calculation

Safety factor against the risk of liquefaction can be defined as the ratio of undrained shear cyclic stress τ_{rc}, calculated from laboratory or in situ tests. The shear stress induced by the seismic solicitation could be evaluated by calculation of dynamic response or by simplified way.

Usually, the required safety factor values in a liquefaction case Fs= τ_{rc} / τ_{sism} varies from 1.3 to 1.5; where the estimation of in-situ ratio $(\tau_{rc} / \sigma'_v)_{in\text{-}situ}$ can be made by means SPT data tests. Curves showing $(\tau_{rc} / \sigma'_v)_{in\text{-}situ}$ function of N'_{SPT} for 3 earthquake magnitudes M (6, 7.5 et 8.25) are given in Seed (1979), where N'_{SPT}: number of shots corrected in order to eliminate the influence of the depth is calculated as $N'_{SPT} = C_N N_{SPT}$, with C_N, standard ratio of SPT test (Liao & Whitman, 1986):

$$C_N = \sqrt{\frac{1}{\sigma_v'}} \quad (\sigma'_v\text{: vertical effective stress in kg/cm}^2) \quad (1)$$

On the other hand, shear cyclic stress τ_{sism} can be estimated from the simplified method developed by Seed & Idriss (1971): $\tau_{sism} = 0.65\, C_D\, [(\gamma h/g)\, a_{max}]$, with:

C_D: reducing factor, depending on depth, (Seed & Idriss, 1971)

γ :specific weight of the ground

h: depth and g: acceleration of gravity

a_{max}: maximum acceleration of ground surface induced by the earthquake, estimated by Seed et al. (1969) according to the seismic magnitude and the epicentre distance.

Algiers Containers Terminal analysis using SPT data

Table 1 shows the average soil characteristics after the earthquake and the earthquake data. Table 2 shows SPT test data for the five soundings performed at the same points of the boreholes S.1 to S.5. Shear cyclic stress τ_{rc} is calculated for two magnitudes M=6 and M=7.5. The results are summarized in table 3. τ_{sism} is calculated for two accelerations: a_{max}=0,064g and a_{max}=0,1g. It should be noted that in the case of M=7 and a d=60km: a_{max} is equal to 0.064g . The results are summarized in table 4.

Table 1- Average soil characteristics after the earthquake and the earthquake data

Dry density γ_d/γ_w	Water content at saturation w_{sat} (%)	Saturated Density γ_{sat}/γ_w	Epicentre distance (Km)	Magnitude (Richter scale)
1.3	39	1.8	60	6.8

Table 2 – SPT test data (LEM, 2005)

	S.1				S.2				S.3				S.4				S.5			
Depth (m)	N1	N2	N3	NSPT	N1	N2	N3	NSPT	N1	N2	N3	NSPT	N1	N2	N3	NSPT	N1	N2	N3	NSPT
0.0-3.0	8	4	3	7	5	2	2	4	4	3	2	5	18	5	2	7	3	1	1	2
3.0-4.5	6	5	4	9	3	2	4	6	4	4	5	9	2	3	1	4	5	8	5	13
4.5-6.0	3	4	2	6	3	6	6	12	6	7	4	11	2	3	4	7	4	3	2	5
6.0-7.5	2	32	2	5	1	2	4	6	3	10	13	23	1	3	2	5	5	2	3	5
7.5-9.0	2	3	3	6	5	6	1	7	5	8	12	22	3	2	4	6	4	6	5	11
9.0-10.5	6	8	12	20	3	4	2	6	4	3	6	9	3	2	2	4				
10.5-12.0					4	5	6	11	1	2	2	4								

$N_{SPT} = N_2 + N_3$

Table 3 – Shear cyclic stress calculated for M=6 and M=7.5

boreholes	z(m)	Z avg (m)	σ'v(kPa)	N_{SPT}	C_N	N'_{SPT}	(τ_{rc} /σ'v) (M=7.5)	τ_{rc} (kPa) (M=7.5)	(τ_{rc}/σ'v) (M=6)	τ_{rc} (kPa) (M=6)
S.1	0-3	1,5	12	7	2,89	20,21	0,21	2,57	0,28	3,32
	3-4.5	3,75	30	9	1,83	16,43	0,18	5,31	0,23	6,82
	4.5-6	5,25	42	6	1,54	9,26	0,10	4,40	0,13	5,67
	6-7.5	6,75	54	5	1,36	6,80	0,08	4,17	0,10	5,48
	7.5-9	8,25	66	6	1,23	7,39	0,08	5,54	0,11	7,23
	9-10.5	9,75	78	20	1,13	22,65	0,24	18,70	0,31	24,15
S.2	0-3	1,5	12	4	2,89	11,55	0,13	1,54	0,16	1,98
	3-4.5	3,75	30	6	1,83	10,95	0,12	3,68	0,16	4,72
	4.5-6	5,25	42	12	1,54	18,52	0,20	8,30	0,25	10,68
	6-7.5	6,75	54	6	1,36	8,16	0,09	5,01	0,12	6,49
	7.5-9	8,25	66	7	1,23	8,62	0,10	6,45	0,13	8,34
	9-10.5	9,75	78	6	1,13	6,79	0,08	6,02	0,10	7,91
	10.5-12	11,25	90	11	1,05	11,60	0,13	11,62	0,17	14,90
S.3	0-3	1,5	12	4	2,89	11,55	0,13	1,54	0,16	1,98
	3-4.5	3,75	30	6	1,83	10,95	0,12	3,68	0,16	4,72
	4.5-6	5,25	42	12	1,54	18,52	0,20	8,30	0,25	10,68
	6-7.5	6,75	54	6	1,36	8,16	0,09	5,01	0,12	6,49
	7.5-9	8,25	66	7	1,23	8,62	0,10	6,45	0,13	8,34
	9-10.5	9,75	78	6	1,13	6,79	0,08	6,02	0,10	7,91
	10.5-12	11,25	90	11	1,05	11,60	0,13	11,62	0,17	14,90
S.4	0-3	1,5	12	5	2,89	14,43	0,16	1,89	0,20	2,42
	3-4.5	3,75	30	9	1,83	16,43	0,18	5,31	0,23	6,82
	4.5-6	5,25	42	11	1,54	16,97	0,18	7,66	0,23	9,84
	6-7.5	6,75	54	23	1,36	31,30	0,35	18,63	0,45	24,10
	7.5-9	8,25	66	22	1,23	27,08	0,29	19,12	0,38	24,76
	9-10.5	9,75	78	9	1,13	10,19	0,11	8,94	0,15	11,49
	10.5-12	11,25	90	4	1,05	4,22	0,05	4,14	0,06	5,78
S.5	0-3	1,5	12	2	2,89	5,77	0,07	0,78	0,09	1,04
	3-4.5	3,75	30	13	1,83	23,73	0,25	7,54	0,32	9,75
	4.5-6	5,25	42	5	1,54	7,72	0,09	3,68	0,11	4,79
	6-7.5	6,75	54	5	1,36	6,80	0,08	4,17	0,10	5,48
	7.5-9	8,25	66	11	1,23	13,54	0,15	9,81	0,19	12,57

Table 4 – Cyclic shear stress calculated for two a_{max} values

Average depth (m)	σ_v (kPa)	a_{max}/g	C_D	τ_{sism} (kPa) a_{max}=0,064g	τ_{sism}(kPa) a_{max}=0,1g
1,5	27	0,064 or 0,1	0,99	1,11	1,74
3,75	67,5	0,064 or 0,1	0,96	2,70	4,21
5,25	94,5	0,064 or 0,1	0,95	3,73	5,84
6,75	121,5	0,064 or 0,1	0,94	4,75	7,42
8,25	148,5	0,064 or 0,1	0,92	5,68	8,88
9,75	175,5	0,064 or 0,1	0,9	6,57	10,27
11,25	202,5	0,064 or 0,1	0,88	7,41	11,58

Figure 4 gather for 2 boreholes S1 & S2 the evolution of the safety factor function of the depth with M=6 and M=7.5 and two accelerations a_{max}=0,064g and a_{max}=0,1g. It appears that the safety factor is often lower than 1.5 in the zones of 5m to 8m depth. This confirms the fact that Algiers container terminal embankment takes part of the family of potentially liquefiable soil.

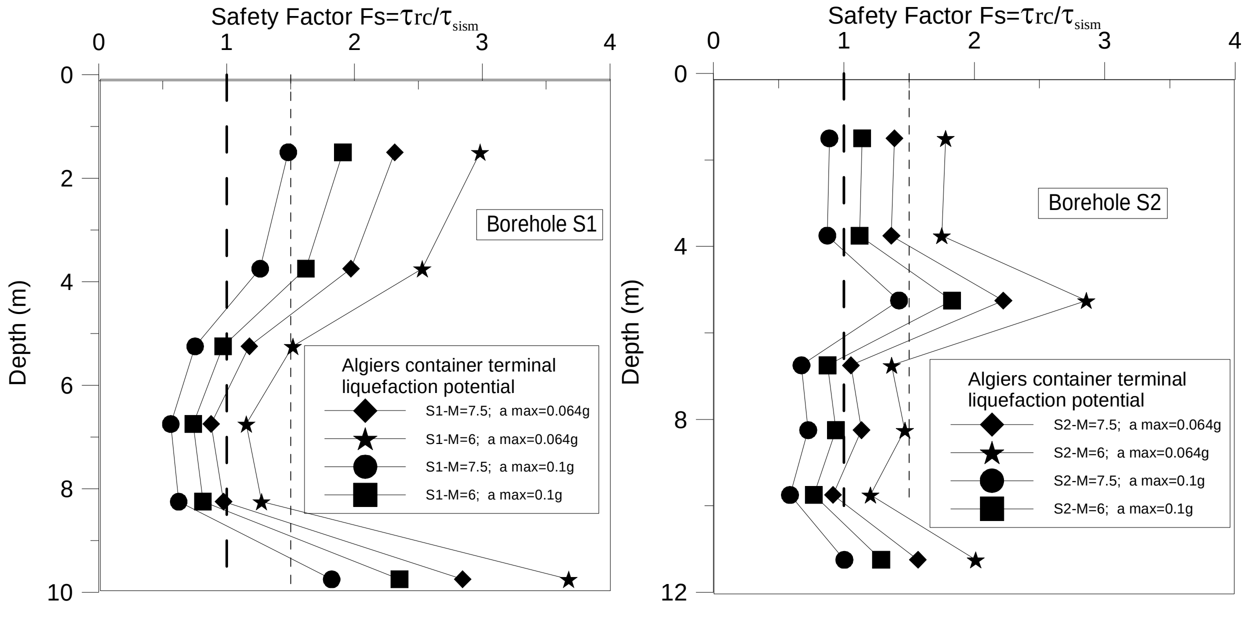


Figure 4. Evolution of the safety factor function of the depth

Evaluation of cyclic liquefaction potential using cone penetration tests (CPT)

Usually, standard penetration test (SPT) is the more common method used for evaluation of liquefaction potential. However data obtained from the cone penetration test (CPT), because of its greater repeatability and continuity of the profile, became even popular these last decades. Number CPT based methods for cyclic liquefaction evaluation can be found in the literature. A rapid description of different methods used for the assessment of soil potential liquefaction under cyclic loadings using CPT soundings is given here below.

CPT calculation method

Resistance to cyclic loading could be illustrated by a cyclic stress ratio able to generate cyclic liquefaction. It is noted cyclic resistance ratio (CRR) and expressed as: $(\tau_{rc} / \sigma'_v)_{in\text{-}situ}$. The safety factor is expressed in this case by : Fs=CRR/ $CSR_{7.5}$, where ratio $(\tau_{sism} / \sigma'_v)$, noted CSR (Cyclic Stress Ratio). The later is calculated via $CSR_{7.5}$ with a magnitude M=7.5 as $CSR_{7.5}$ =CSR/MSF. MSF is a Magnitude scale factor : $MSF=(M/7.5)^{-2.56}$.

Robertson method (1998)

Several correlations using corrected CPT tip resistance have been proposed for the assessment of cyclic resistance ratio (CRR) for clean sands and silty sands (e.g. Robertson and Wride 1998; Olsen and Koester 1995; Robertson and Fear 1995)

$$
\begin{aligned}
CRR &= 0.833\left[\frac{q_{c1N,CS}}{1000}\right] + 0.05 \qquad \text{si } q_{c1N,CS} < 50 \\
CRR &= 93\left[\frac{q_{c1N,CS}}{1000}\right]^3 + 0.08 \qquad \text{si } 50 \le q_{c1N,CS} < 160
\end{aligned}
\tag{2}
$$

Where $q_{c1N,CS}$ is the equivalent clean sand normalized (to vertical stress=100kPa) CPT penetration resistance. The flow chart proposed by Robertson and Wride (1998) allows estimation of $q_{c1N,CS}$ and thus calculation of CRR.

Olsen method (1997)

This method gives CRR value as rapidly described here below (Olsen, 1997).

$$
CRR = 0.00128\left[\frac{q_c}{(\sigma'_v)^{0.7}}\right] - 0.025 + 0.17R_f - 0.028R_f^2 + 0.0016R_f^3 \tag{3}
$$

It is function of tip resistance (q_c in atm), effective vertical stress (σ'_v in atm) and friction ratio R_f (%) where:

$R_f = (f_s/q_c) * 100$ and f_s : CPT sleeve friction stress

Juang method (2003)

In Juang CPT based method (Juang et al., 2003), soil liquefaction resistance, noted CRR, is calculated as:

$$
CRR = C_\sigma \exp\left[-2.957 + 1.264\left(\frac{q_{c1N,CS}}{100}\right)^{1.25}\right] \tag{4}
$$

Where:

$$
C_\sigma = -0.016\left(\frac{\sigma'_v}{100}\right)^3 + 0.178\left(\frac{\sigma'_v}{100}\right)^2 - 0.063\left(\frac{\sigma'_v}{100}\right) + 0.903 \qquad \text{with } \sigma'_v \text{ (kPa)}
$$

$$
q_{c1N,CS} = q_{c1N}\left(2.429I_c^4 - 16.943I_c^3 + 44.551I_c^2 - 51.497I_c + 22.802\right)
$$

$$
q_{c1N} = = \left[\frac{q_c}{(\sigma'_v)^{0.5}}\right] \qquad \text{with } q_c \text{ et } \sigma'_v \text{ (atm)}
$$

The effect of soil type on CRR is considered using a soil type index modified by Juang from Lunne et al.,1997)

$$
I_c = \left[(3.47 - \log q_{c1N})^2 + (\log F + 1.22)^2\right]^{0.5}
$$

Where normalized friction ratio F is calculated by:

F= $(f_s/(q_c-\sigma'_v)$ *100   (Lunne et al. , 1997)

Algiers Containers Terminal analysis using CPT data

For Algiers container terminal case, eleven test data were chosen for liquefaction evaluation. Their site positions lies on the whole terminal platform area. CPT data of points N°1, 2, 3, 16, 18, 20, 26, 30, 32, 36, 41 (figure 2b) are taken into account. Safety factors results presented here below come from deterministic approach proposed by Olsen method (Olsen, 1997) as well as Juang method (Juang et al. 2003). CRR and CSR are calculated for the different chosen points subjected to an earthquake loading with maximum horizontal ground surface acceleration limited to 0.1g.

Safety factor F_S = CRR/CSR distribution for the different points are calculated using both the two method (figure 5). Figures include also application of the Juang proposed probabilistic method for estimation of mapping function (P_L), with:

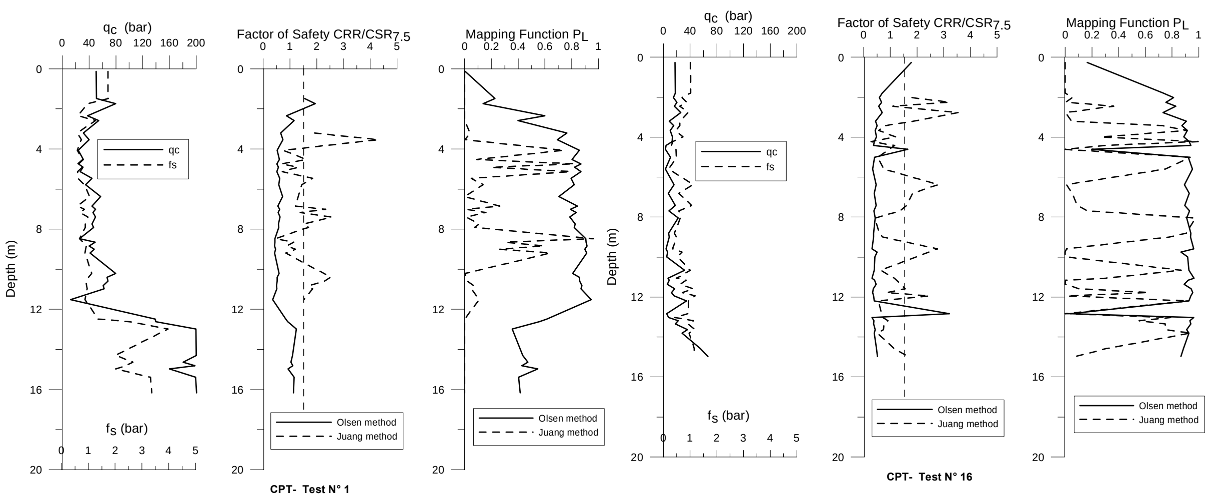
$P_L = 1 / [1+(F_S /0.96)^{4.5}]$ (Juang et al, 2003)

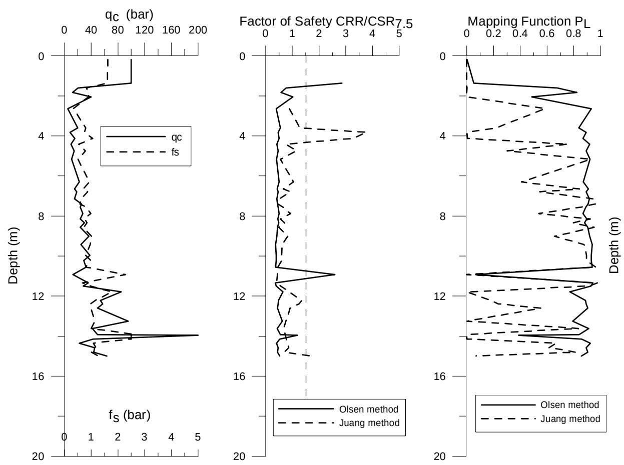
Where P_L is proposed as likelihood of liquefaction parameter given in table 5

Series of curves presented here below (figure 5) show results of the present site soil state calculation for Algiers Container Terminal platform vis-à-vis potential liquefaction. The Juang method seems to give results more dispersed than that of Olsen method. However, great part of the tests shows high liquefaction risk.

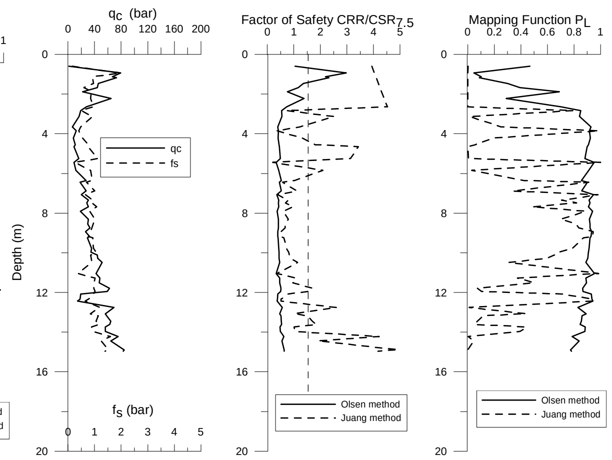
Table 5 – Likelihood liquefaction (Chen and Juang, 2000)

Probability	Description
$0.85 \le P_L < 1.00$	Almost certain that it will liquefy
$0.65 \le P_L < 0.85$	Very likely
$0.35 \le P_L < 0.65$	Liquefaction/non liquefaction equally likely
$0.15 \le P_L < 0.35$	Unlikely
$0.00 \le P_L < 0.15$	Almost certain it will not liquefy

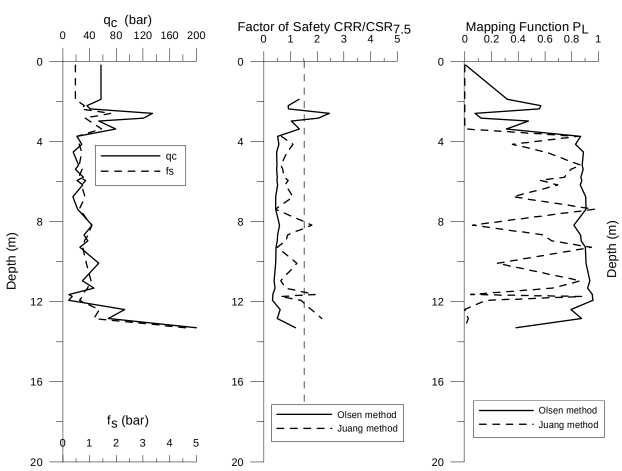




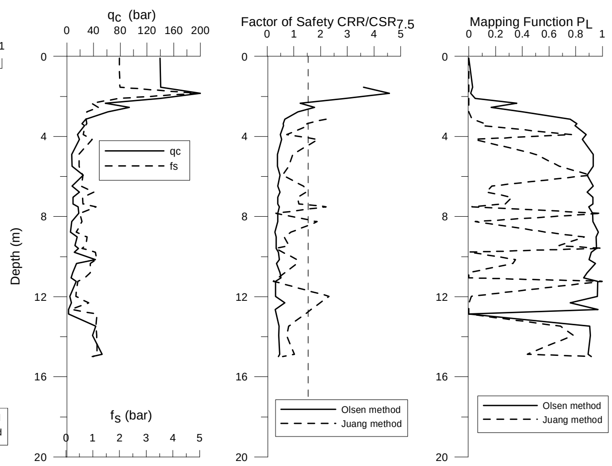
CPT- Test N° 2



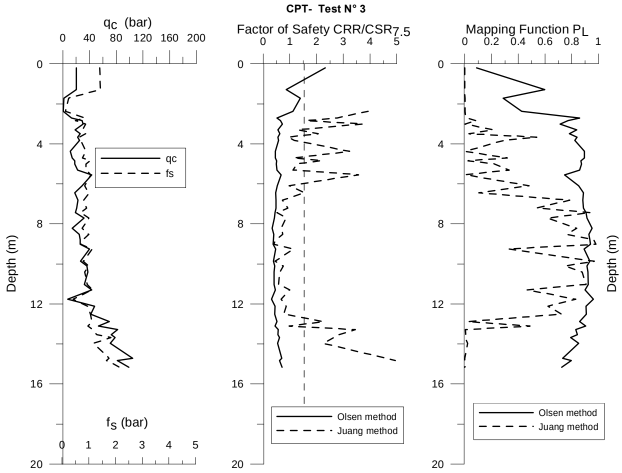
CPT- Test N° 18



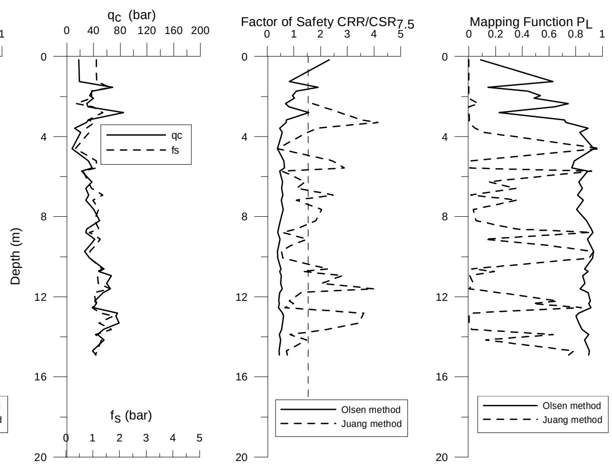
CPT- Test N° 3



CPT- Test N° 20



CPT- Test N° 26



CPT- Test N° 36

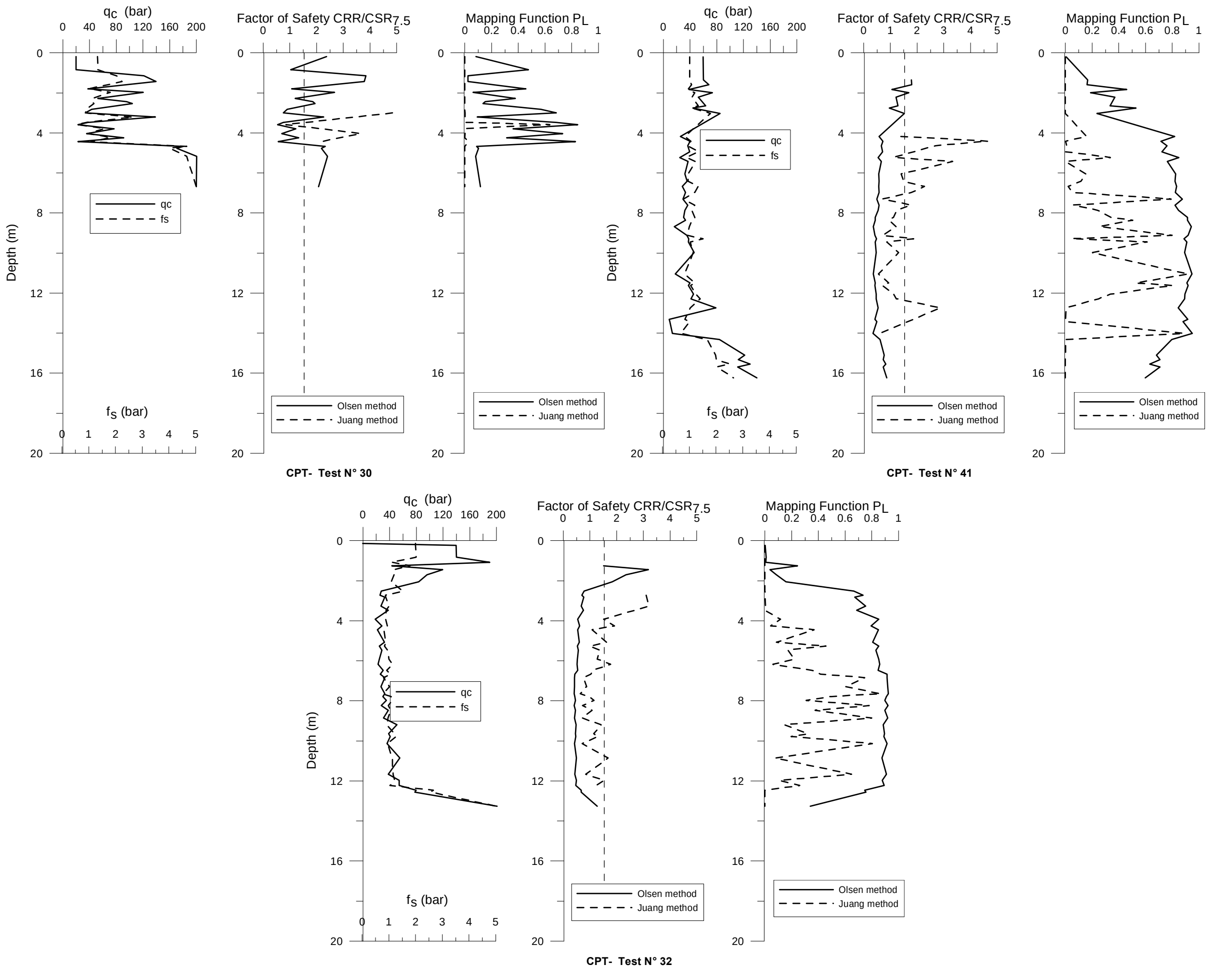


Figure 5. CPT results for the different chosen points

Comparison between SPT and CPT potential liquefaction results

One of the present paper aims is an analysis of results given by both SPT as well as CPT based methods using neighbored data sources. Comparison of safety factor calculated from SPT test data S1 and CPT test data N°3 and those from S2 and CPT N°41 (figure 2b) using both Olsen and Juang methods is thus presented in figure 6. Choice of these data points is made because of their close respective positions. If we compare safety factors determined using SPT as well as CPT soundings, results seem coherent, especially in the ground layers 4-12m for S2 and 4-9m for S1. SPT Fs profile appears between the CPT Olsen one and that of Juang. Good agreement has to be noted for Fs (SPT) and Fs (CPT Olsen) in 6-10m for S2 and 6-8m for S1. It corresponds to critical depths around 6m where generally occurs the liquefaction phenomena.

The method of Olsen seems to give liquefaction safety factors weaker than that of Juang. The latter is quasi-constant around 0.5 between 4-12m ground depth, whereas the method of Juang presents more important fluctuations into these layers. Nevertheless, the two methods give values of safety coefficient lower than 1.5.

CONCLUSIONS

First analysis using grading curves of Algiers container terminal sands shows their position located into a potentially liquefiable sand spindle. SPT tests results revealed that the relative in situ soil density remains very low. The risk analysis based on SPT tests, figures 4.1 to 4.5 clearly confirms the likelihood of soil liquefaction for the two prospected magnitudes and two accelerations. Safety factors below 1.3 to 1.5 for earthquake magnitude higher than 6 appear in the different soundings, in particular in fairly deep soil layers from 5 to 8 m. This risk is largely confirmed by the analysis using CPT data tests (Figure 5). Comparison of safety factors determined using SPT as well as CPT soundings gives good agreement. The method of Olsen seems to give liquefaction safety factors weaker than that of Juang. The latter is quasi-constant around 0.5 between 4-12m ground depth, whereas the method of Juang presents more important fluctuations into these layers. Nevertheless, the two methods give values of safety coefficient lower than 1.5. Finally, somewhat is the method of interpretation used, the two tests in-situ CPT and SPT highlight a real potential of liquefaction in the current state of Algiers container terminal ground. It seems necessary to carry out a particular treatment of this liquefiable site by seeking solutions to increase the cyclic resistance of the sediment. This increase can be obtained by in situ ground densification so as to reach relative densities higher than 80% using various techniques such as vibratory floatation, dynamic compaction, high pressure injections techniques or improvement of the drainage.

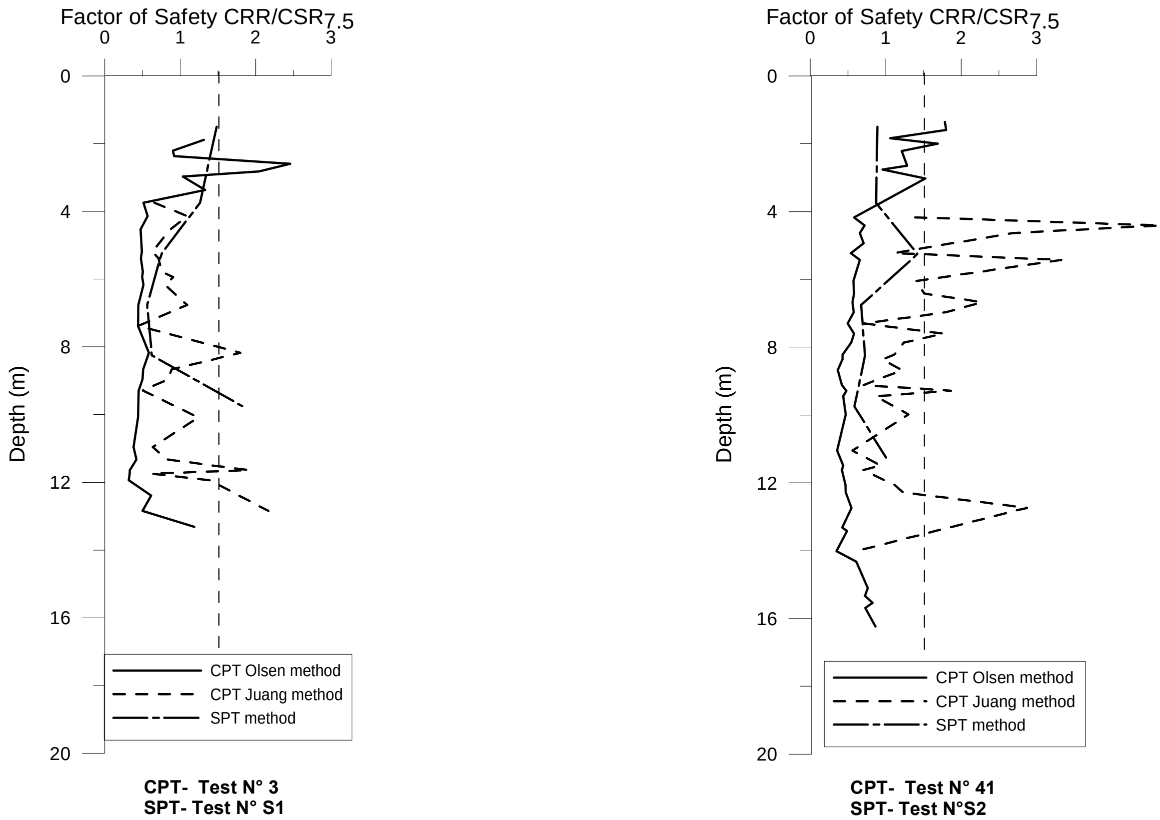


Figure 6. Comparison between SPT and CPT results

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