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Evaluation of liquefaction potential by CPTU and SDMT

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DMT 15

IN SITU SITE INVESTIGATION

ABSTRACT

This paper deals with the evaluation of liquefaction potential of the soils at the marsh of Porto Romano, located in the western central part of Albania, where will be constructed the Energetic Park of Porto Romano. These analyses are performed based on the data taken from different in situ testing techniques. Piezocone tests (CPTU) were carried out during the two different site investigations (before and after the ground improvement) for the characterization of soil layers and determination of soil properties. In addition, during the first phase of site investigation a full seismic study was performed to estimate the shear wave velocity, V_s, and the peak ground acceleration. During the second site investigation two seismic dilatometer Marchetti tests (SDMT) were also carried out very close to the CPTU locations. The paper shows the results of the liquefiability assessment by CPTU and SDMT tests carried out during the site investigations.

Keywords: *liquefaction, Piezocone test (CPTU), Seismic Dilatometer Marchetti (SDMT)*

INTRODUCTION

The most used methods to assess the liquefaction potential are based on CPTU or SDMT data to calculate the cyclic resistance ratio, *CRR* and the cyclic stress ratio, *CSR*.

This paper aims to deal with the assessment of liquefaction potential of a construction site in Albania.

The site is classified as Category III according Albanian Earthquake Design Regulation KTP-N.2-89 (1989). The peak ground acceleration is 0.39g and based on the seismic study, the maximal earthquake magnitude for this construction site is 6.8.

The methods of Robertson (2009) and Boulanger & Idriss (2014) are used to calculate the factor of safety against the triggering of liquefaction, engaging the cone resistance, q_t, adjusted by a correction factor, K_C, that takes into consideration the fine content and soil plasticity. The calculations employ iterative approaches to determine *CRR* based on the data taken by CPTU (Robertson & Wide 1998). The cyclic stress ratio, *CSR*, is estimated based on Seed & Idriss (1971).

SDMT tests provide two very important parameters for evaluating the liquefaction potential, the horizontal stress index, K_D, and the shear wave velocity, V_s. Evaluation of liquefaction resistance at each test depth can be obtained using CRR-K_D and CRR-V_S correlations. The SDMT data engaged in these calculations include the horizontal stress index, K_D, and hence the stress history, into CPTU liquefaction correlations in order to reduce the uncertainty of *CRR* estimated by CPTU (Marchetti, 2014). *CRR* will be estimated from correlations, using at the same time Q_{cn} and K_D.

The developed SDMT based methods are very good complementary to the CPTU based methods for liquefaction analysis.

METHODOLOGIES FOR ASSESSING THE LIQUEFACTION POTENTIAL

The factor of safety: $FoS = \left(\frac{CRR}{CSR}\right) \cdot MSF$

CSR calculations: $CSR_{7.5} = \frac{\tau_{av}}{\sigma'_{v0}} = 0.65 \cdot \left(\frac{\sigma_{vo} \cdot a_{max}}{\sigma'_{v0}}\right) \cdot r_d$

-Robertson (2009)

$r_d = 1 - 0.00765z$, if z < 9.15 m
$r_d = 1.174 - 0.0267z$, if 9.15 < z < 23 m
$r_d = 0.744 - 0.008z$, if 23 < z < 30 m
$r_d = 0.5$, if z > 30 m

Boulanger & Idriss (2014)

$\ln(r_d) = \alpha(z) + \beta(z) \cdot M$, if z ≤ 34 m
$\alpha(z) = -1.012 - 1.126 \cdot \sin\left(\frac{z}{11.73} + 5.133\right)$
$\beta(z) = 0.106 + 0.118 \cdot \sin\left(\frac{z}{11.28} + 5.142\right)$

CRR calculations from CPTU: For M = 7.5
-Robertson (2009): using initial stress exponent *n = 1.0*; Q_{tn}, F_R & I_c.

The stress exponent: $n = 0.38 \cdot I_c + 0.05 \cdot \left(\frac{\sigma'_{v0}}{P_a}\right) - 0.15$, $n \le 1.0$

where: $C_N = \left(\frac{P_a}{\sigma'_{v0}}\right)^n$ Iteration until the change in n produces $\Delta n \le 1$ is required.

$CRR_{7.5} = 93 \cdot \left(\frac{Q_{tn,cs}}{1000}\right)^3 + 0.08$, if $50 \le Q_{tn,cs} \le 160$ where: $Q_{tn,cs} = K_C \cdot Q_{tn}$

-Boulanger & Idriss (2014): considering cone resistance and involving the value of CN:

$C_N = \left(\frac{P_a}{\sigma'_{v0}}\right)^{\beta} \le 1.70$ where: $\beta = 1.338 - 0.249 \cdot (q_{c1N})^{0.264}$ where: $q_{c1N} = C_N \cdot q_c$

The equivalent clean sand of CPT: $q_{c1N,cs} = q_{c1N} \cdot \Delta q_{c1N}$

where: $\Delta q_{C1N} = (11.9 + \frac{q_{c1N}}{14.6}) \cdot \exp(1.63 - \frac{9.7}{FC+2} - \left(\frac{15.7}{FC+2}\right)^2$

$$CRR_{7.5} = \exp\left\{\frac{q_{c1N,cs}}{113} + \left(\frac{q_{c1N,cs}}{1000}\right)^2 - \left(\frac{q_{c1N,cs}}{147}\right)^3 - \left(\frac{q_{c1N,cs}}{137}\right)^4 - 2.8\right\}$$

CRR calculations from SDMT: K_D - CRR
-Robertson (2012): $CRR = 93 \cdot (0.025 \cdot K_D)^3 + 0.08$

-B&I (2006): $CRR_{7.5} = \exp\left\{\frac{Q_{cN}}{540} + \left(\frac{Q_{cN}}{67}\right)^2 - \left(\frac{Q_{cN}}{80}\right)^3 - \left(\frac{Q_{cN}}{114}\right)^4 - 3\right\}$

$$AverageCRR = [(CRRfromQ_{cn}) \cdot (CRRfromK_D)]^{0.5}$$

SITE INVESTIGATIONS AND GROUND IMPROVEMENT

Project description and Site investigations before ground improvement

The project considered in this paper is the Oil Product Storage in Porto Romano, located on a flat area near Durresi city, in Albania. The area is a marsh, at an average initial level of - 0.40 m below the sea level. The project includes a set of oil product storage tankers, LPG tankers, pipelines, etc. The plan of the project facilities and in situ tests is presented in Fig. 1.

The first site investigation included 12 boreholes, where SPT and laboratory tests were carried out. Also 4 CPTU tests, 3 of them down to 25 - 27 m and the last one down to 14 - 15 m.

The results of the SPT and one CPTU tests are presented in Fig. 2 and 3.

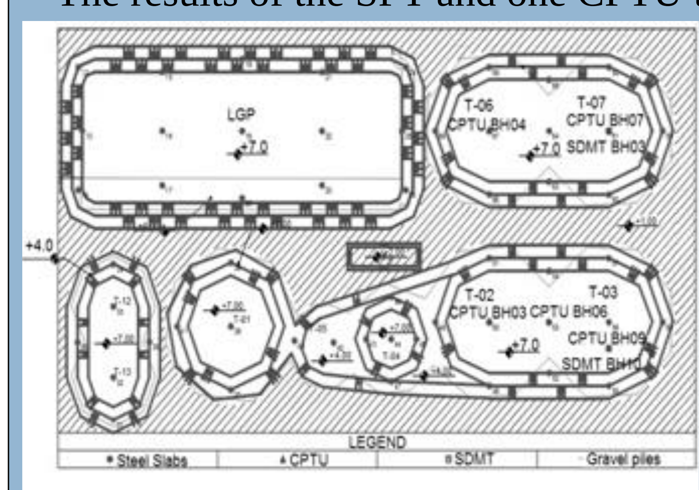

Figure 1 Plan of the construction site.

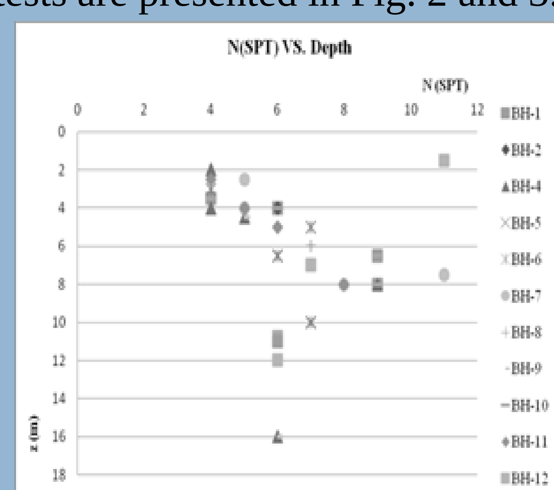

Figure 2 SPTs results.

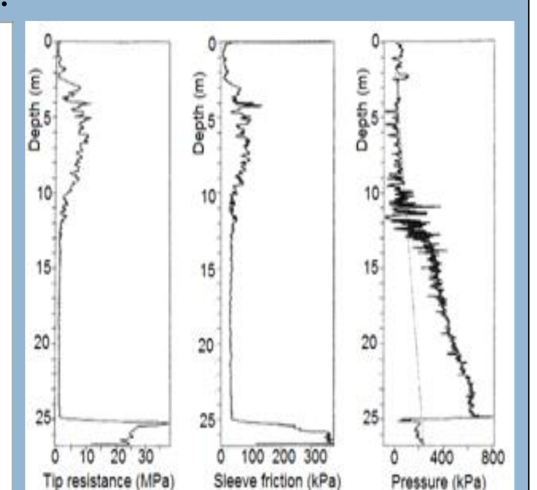

Figure 3 CPTUs results.

Fig. 4 gives the derived parameter of fine content particles (<0.06 mm), based on CPTU data. FC lies between 5% and 15% for the layers between 2 m and 10 m depth and then increases with depth. From CPTU tests measurements were detected that the layers susceptible to liquefaction are loose saturated sands, silty sands or sandy silts, located between 2 m and 15 m depth. The analysis of liquefaction potential is than performed on the basis of Idriss & Boulanger (2004) procedure and some of the results are presented in Fig. 5 (for CPTU 03). (Logar, 2009)

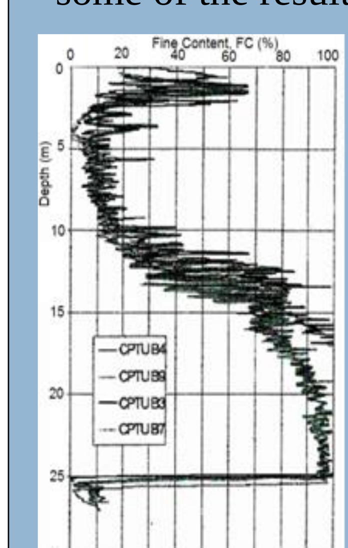

Figure 4 FC from CPTU.

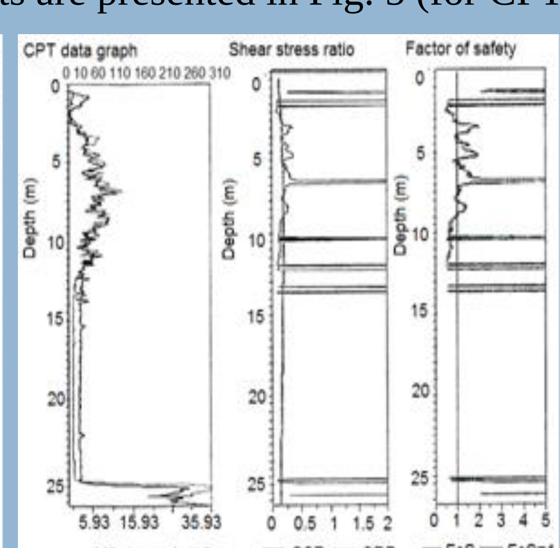

Figure 5 Interpretation on liquefaction potential

The installation of stone columns, should increase the value of cone end resistance, qc as given in Table 1 (Japanese Geotechnical Society, 2006).

Table 1 Increase in qc values as a function of replacement ratio As

Replacement ratio (A_s)	10%	15%	20%
Increase in q_c (Δq_c)	4 MPa	6 MPa	8 MPa

Table 2 Required and achieved cone resistance with 15 % As

Depth (m)	Required q_c (MPa)	New q_c (MPa) with A_s = 15% B3	B4	B7	B9
3	8.5	8.8	13.5	13.9	14.8
4	9.0	9.1	14.4	26.7	34.8
5	9.6	14.9	15.6	22.2	17.5
6	10.2	13.9	13.6	12.8	13.6
7	10.6	13.5	14.1	15.6	12.6
8	10.9	14.5	12.8	14.2	15.6
9	11.3	13.7	12.7	13.2	11.9
10	11.6	11.2	9.7	13.5	13.0
11	12.0	11.5	8.4	10.9	9.7
12	12.3	8.6	7.5	9	9.3

The effect of 15% replacement ratio on the safety against liquefaction is shown in Table 2, including the required q_c to prevent liquefaction and the expected new q_c after executing ground improvement. Safety against liquefaction is achieved until the depth of around 10 m, but not below 10 m. The solution for safety against liquefaction was to replace 15% of the ground by stone columns (Logar, 2009).

Site investigations after ground improvement

The first phase of ground improvement included the construction of an embankment of 1 m, the stone columns with diameter of 80 cm, which were installed until 14 m and 24 m below the embankment surface, and prefabricated vertical wick drains. The results of CPTU and SDMT tests carried out after the first phase of ground improvement are presented in Fig. 6a,b and Fig. 7a,b.

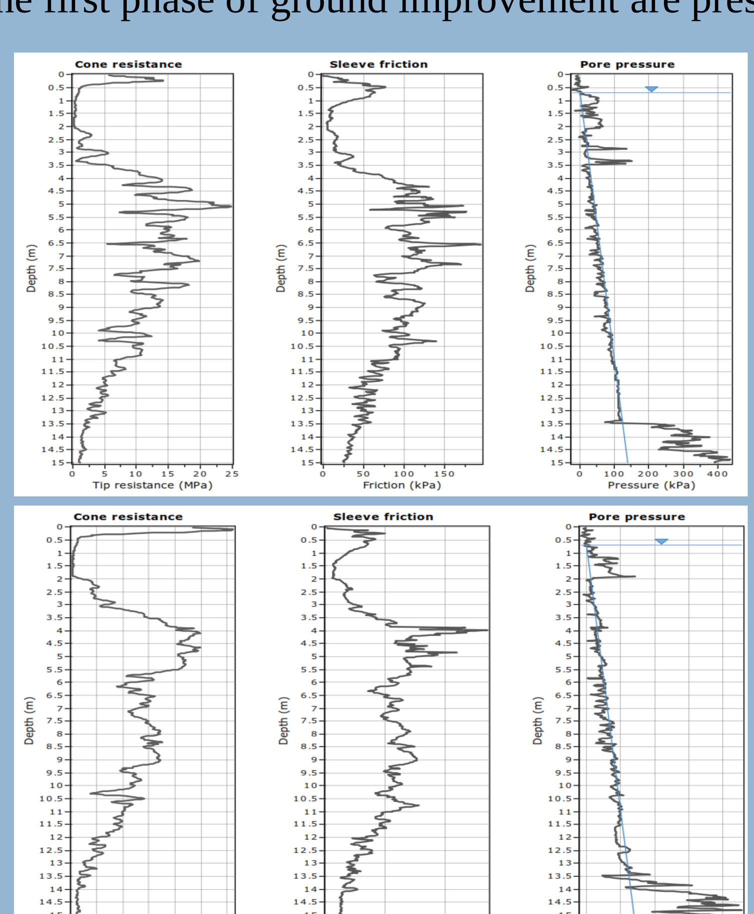

Figure 6 CPTU measurements

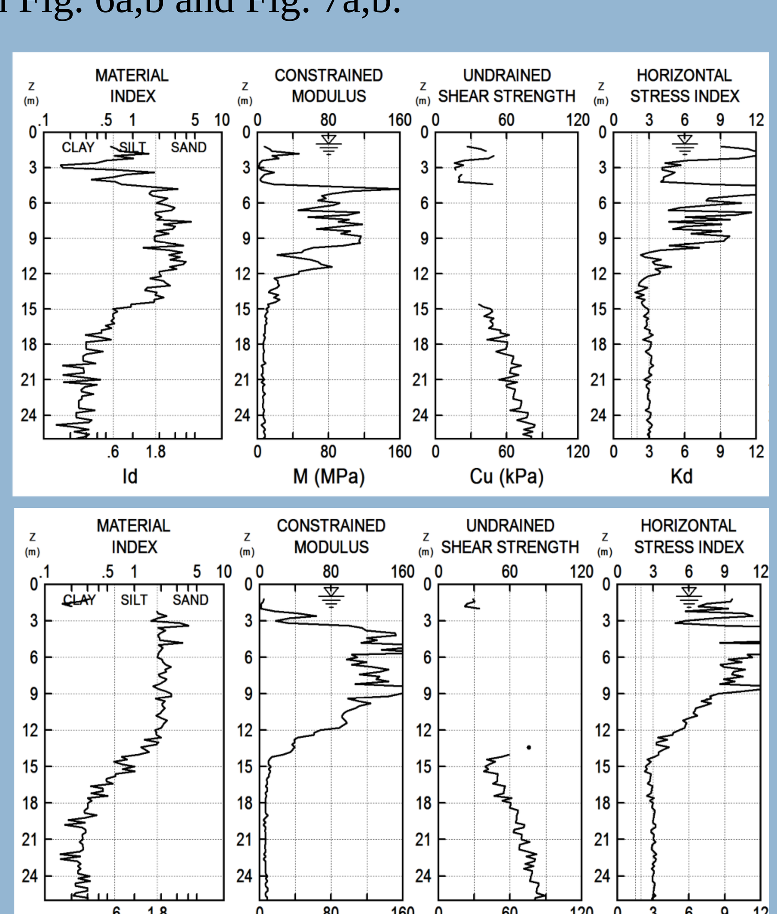

Figure 7 SDMT measurements

RESULTS OF CALCULATIONS

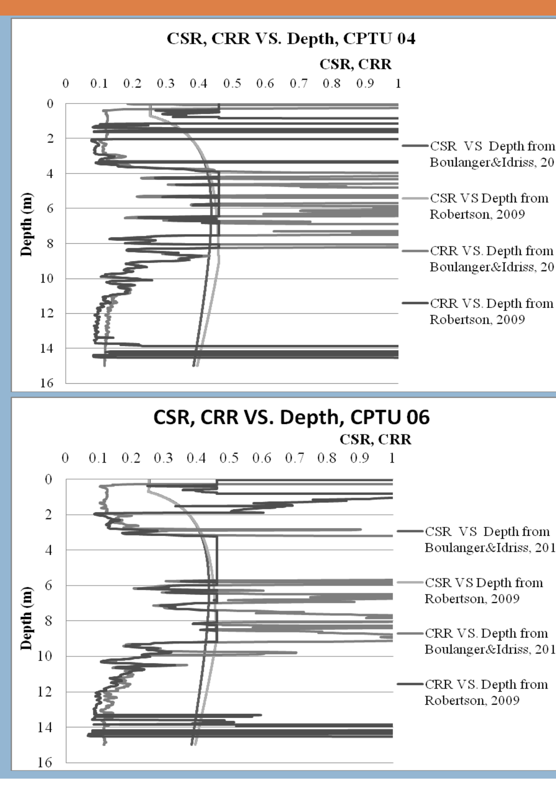

Figure 8 CSR and CRR calculated from CPTU 04 and CPTU 06 data.

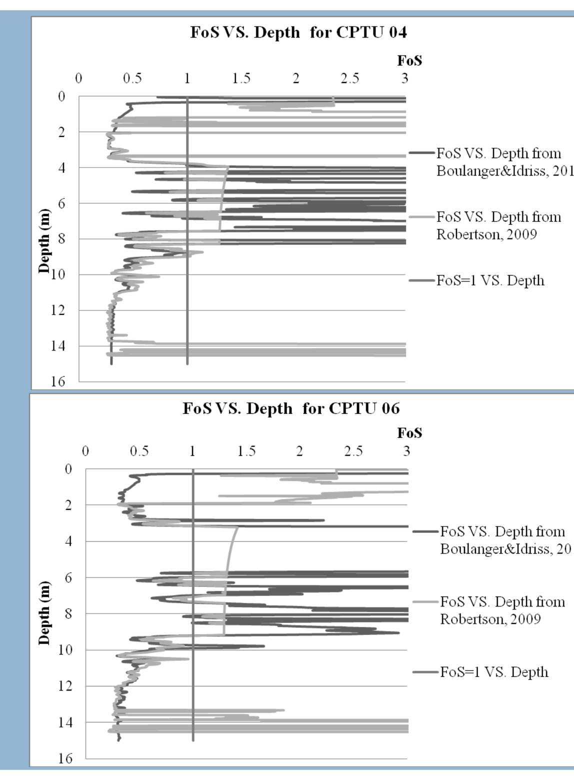

Figure 9 FoS calculated from CPTU 04 and CPTU 06 data.

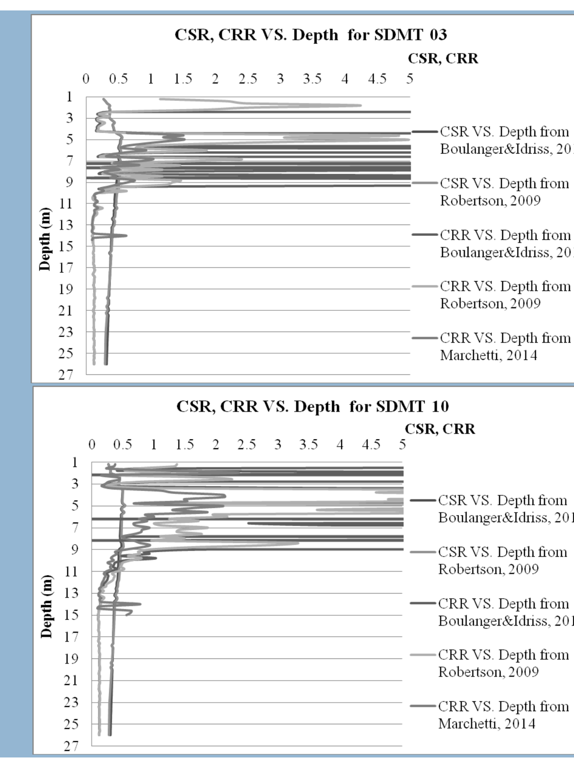

Figure 10 CSR and CRR calculated from SDMT 03 and SDMT 10 data.

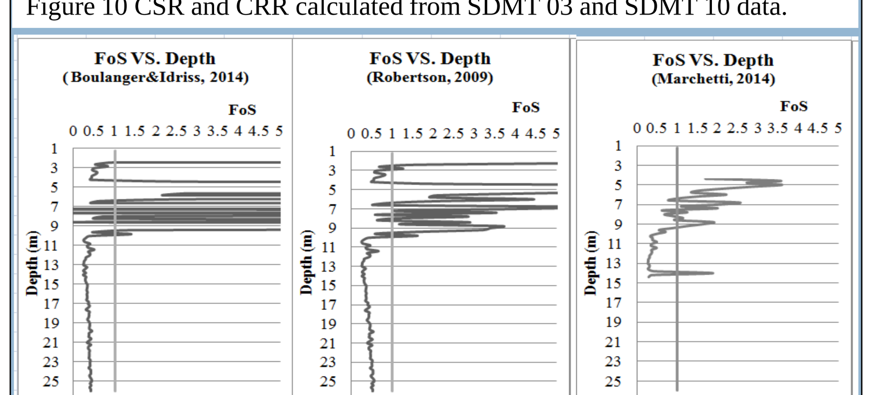

Figure 11 FoS calculated from SDMT 03 data.

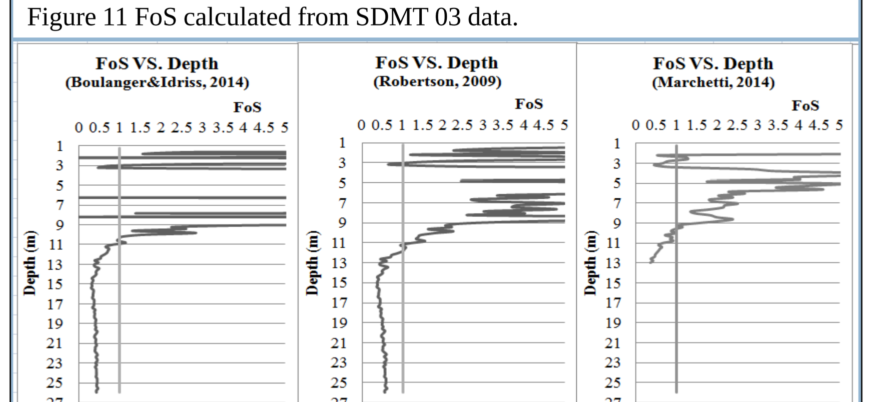

Figure 12 FoS calculated from SDMT 10 data.

CONSLUSIONS

1. The soils susceptible to liquefaction varies from 1 m to 3 m and 9 m to 13.5 m (based on calculations from CPTU data).
2. *CRR* values calculated based on Q_t are in good agreement with the *CRR* values based on K_D.
3. Calculations based on SDMT data, using the relationship CRR – K_D, give the same depths for the layers susceptible to liquefaction. They vary from 2.5 m to 4 m and from 9.5 m to 26 m for SDMT 03 & from 11 m to 26 m, for SDMT 10.
4. The method developed by Marchetti (2014), based on the data measured by both tests is a complementary method to the other ones used.
5. From CPTU 04 & SDMT 03 data, liquefaction is predicted from 8.5 m to 13.5 m. From CPTU 06 & SDMT 10 data liquefaction is predicted from 2 m to 3 m and from 9 m to13 m.
6. Based on the results of this paper, it is better to use both CPTU & SDMT parameters to overpass uncertainties and to get satisfactory reliability in liquefaction potential evaluation.