

# Evaluation of Liquefaction Potential by CPTU and SDMT

Ergys Anamali

Doctoral School, Faculty of Geology and Mines, Tirana, Albania. E-mail: [gysanamali@yahoo.co.uk](mailto:gysanamali@yahoo.co.uk)

Luisa Dhimitri

In Situ Site Investigation, Battle, UK. E-mail: [luisadhimitri@insitusi.com](mailto:luisadhimitri@insitusi.com)

Neritan Shkodrani

Faculty of Civil Engineering, Polytechnic University, Tirana, Albania. E-mail: [neritans@yahoo.com](mailto:neritans@yahoo.com)

Darren Ward

In Situ Site Investigation, Battle, UK. E-mail: [darrenward@insitusi.com](mailto:darrenward@insitusi.com)

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

**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 two site investigations.

## 1 INTRODUCTION

Piezocone Test (CPTU) and Seismic Dilatometer Test (SDMT) are widely recognized as valuable techniques to determine the soils properties in site and the change of stratigraphy. But these tests are also very useful to evaluate the liquefaction potential of soils.

The most used methods assess the liquefaction potential by relating the results of CPTU or SDMT with numerous empirical procedures, in order to compare the cyclic resistance ratio,  $CRR$  with the cyclic stress ratio,  $CSR$ .

This paper aims to deal with the assessment of liquefaction potential of a construction site in Albania, by means of different methods based on CPTU and SDMT data.

The soils in this site, according the seismic study, are classified as Category III after Albanian Earthquake Design Regulation KTP-N.2-89 (1989). The peak ground acceleration is specified to be 0.39g.

The methods of Robertson (2009) and Boulanger & Idriss (2014) are used to calculate the factor of safety against the triggering of liquefaction, by using

the input data from CPTU tests. Cone tip resistance, sleeve friction, the moment magnitude of the earthquake, maximum surface acceleration during earthquake, ground water level and the unit weights are involved in the calculations. The cone resistance,  $q_t$ , adjusted by a correction factor,  $K_C$ , that takes into consideration the fine content and soil plasticity, employ an iterative approach 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). Based on the seismic study, the maximal earthquake magnitude for this construction site is 6.8. The magnitude scaling factor,  $MSF$ , is calculated according to consistent methods with the calculation of  $CSR$ , based on the number of equivalent uniform stress cycles and earthquake magnitude.

Meanwhile, 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 from  $K_D$  and  $V_s$  according to recommended  $CRR-K_D$  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 CPT liquefaction correlations in order to reduce the uncertainty of  $CRR$  estimated by CPTU. (Marchetti, 2014)

At the end of this paper, the  $CRR$  will be estimated from correlations, using at the same time  $Q_{cn}$  and  $K_D$ . The developed SDMT based methods have the potential to be a very good complementary test to the CPTU based methods for liquefaction analysis. As long as  $CRR$  can be evaluated by a correlation based at the same time on both  $Q_{cn}$  and  $K_D$ , many uncertainties related to liquefaction potential evaluation can be over passed.

## 2 SITE INVESTIGATIONS AND GROUND IMPROVEMENT

### 2.1 Site investigations

The project considered in this paper is the Oil Product Storage in Porto Romano, located on a flat area north of Durrësi city, in Albania. The area is a marsh, at an average initial level of - 0.40 m below the sea level. A set of oil product storage tankers, LPG tankers, pipelines, roads and railway system is foreseen to be constructed. The plan of the project facilities and in situ tests carried out during the geotechnical investigations of this project is presented in Fig. 1.

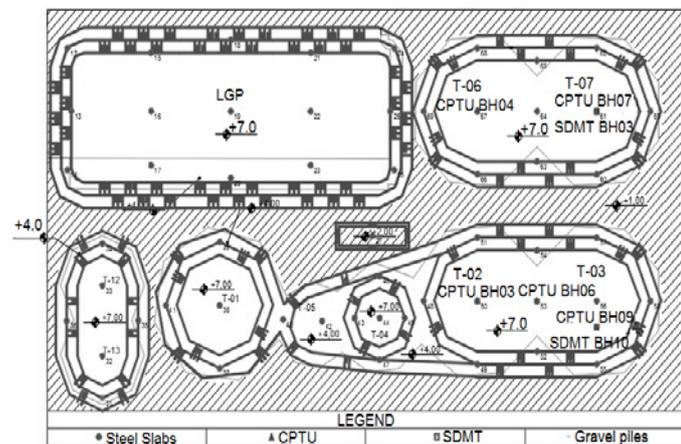


Fig. 1. Plan of the construction site.

The first investigation started in the beginning of year 2009, including 12 boreholes, SPT tests and laboratory tests. Due to very poor quality data taken by a few laboratory tests carried out on very disturbed samples, in September 2009 was decided to carry out 4 CPTU tests. 3 of them were carried out down to 25 - 27 m, until the dense layer of sand was met and 1 of the them was carried out down to 14 - 15 m. Fig. 2 and 3 present the results of SPTs and one of the CPTU tests carried out in that time.

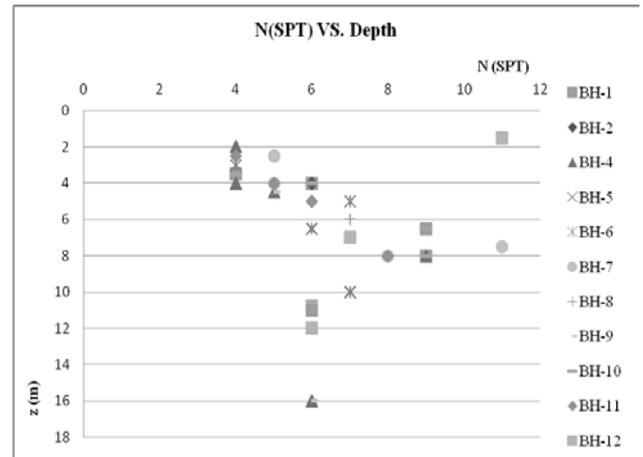


Fig. 2. SPT measurements.

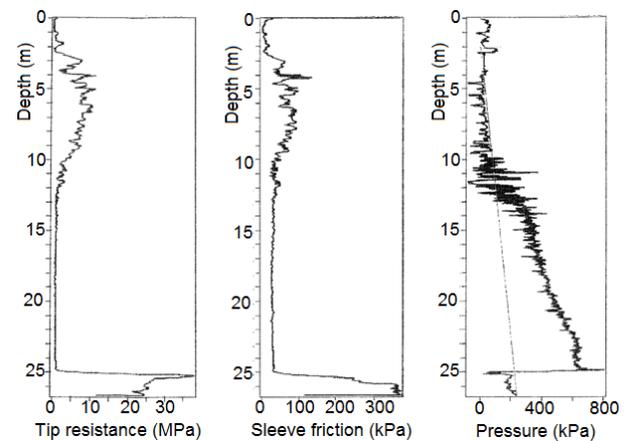


Fig. 3. CPTU 04 measurements.

Fig. 4 gives the derived parameter of fine content particles ( $<0.06$  mm),  $FC$  based on CPTU measurements.  $FC$  lies between 5% and 15% for the layers between 2 m and 10 m depth and then increases with depth.

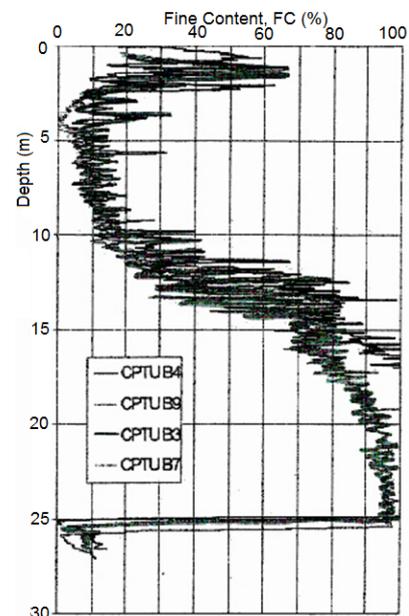


Fig. 4. Fine content, FC (%) derived from CPTU.

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 taking into account firstly the fill layer at the top and some of the results are presented in Fig. 5. (Logar, 2009)

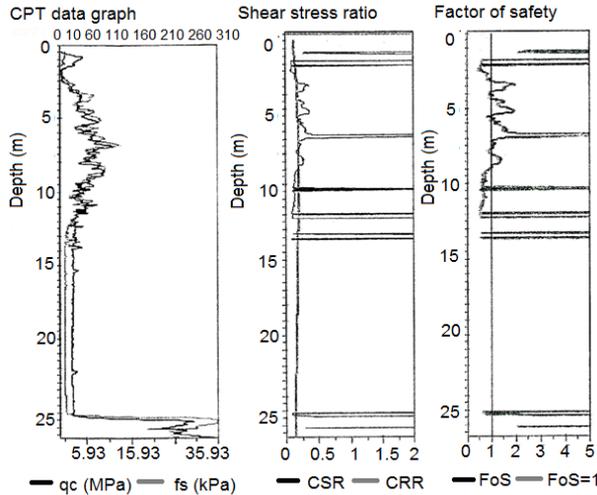


Fig. 4. Interpretation on liquefaction potential, CPTU 03.

Due to the high liquefaction potential, during the second analysis was considered also the effect of the installation of stone columns, by means of the replacement ratio,  $A_s$ , defined as the ratio of cross sectional area of one pile with the total cross sectional area of the grid corresponding to this pile. In order to avoid liquefaction, the installation of stone columns, should increase the value of cone end resistance,  $q_c$  as a function of the replacement ratio,  $A_s$ , as given in Table 1 (Japanese Geotechnical Society, 2006).

Table 1. Increase in  $q_c$  values as a function of replacement ratio  $A_s$

| Replacement ratio ( $A_s$ )        | 10%   | 15%   | 20%   |
|------------------------------------|-------|-------|-------|
| Increase in $q_c$ ( $\Delta q_c$ ) | 4 MPa | 6 MPa | 8 MPa |

The demonstration of effect of 15% replacement ratio on the safety against liquefaction is represented in Table 2, where are shown the values of required  $q_c$  to prevent liquefaction. Also the values of the expected new  $q_c$  after executing ground improvement for achieving 15% replacement ratio for each CPTU; B3, B4, B7, B9, which procures sufficient safety against liquefaction until the depth of around 10 m, but is still too low at the depths below 10 m.

The proposed solution for safety against liquefaction was decided to replace 15% of the

ground by stone columns under all major structures on the oil terminal. In the geotechnical report, it was recommended to install the stone columns down to 12 - 13 m, made by crushed stone particles with a diameter 16 mm - 32 mm (Logar, 2009).

Table 2. Required and achieved cone resistance with 15% replacement ratio

| 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  |

## 2.2 Ground improvement

In order to prevent liquefaction and differential settlements of oil storage tankers and other facilities of the project, the first phase of ground improvement consisted in the construction of an embankment, with the height of 1 m, in the area of railway system and access roads. In the area of tankers and other oil products storage places, in order to prevent the liquefaction the stone columns were executed, with diameter of 80 cm, installed until 14 m of depth, measured from the surface of the embankment. The axial distance between them is 1.8 m below the oil tankers and 2.0 m below the area of the LGP tanker.

To accelerate the consolidation process in the area below the tankers, another system of vertical wick drains was executed, that was installed until 24 m below the embankment surface. The prefabricated vertical wick drains have a width of 10 cm and thickness of 0.5 cm and they are placed in site in the same axial distance as the stone columns. The water coming out from the vertical drainage system is collected by a horizontal drainage system and sent out of the construction site.

At the end of this phase of ground improvement, 4 CPTU tests and 2 SDMT tests are carried out, in order to control the results of the first phase of ground improvement and also to decide for the second and the third phase of ground improvement, by adding other 3 m and 4 m of filling material, respectively, over the old embankment of 1 m.

2 CPTU tests, CPTU 04 and CPTU 06 are used in this paper to assess the liquefaction potential at the end of the first phase of ground improvement. The

results of these tests are presented in Fig. 5a and Fig. 5b.

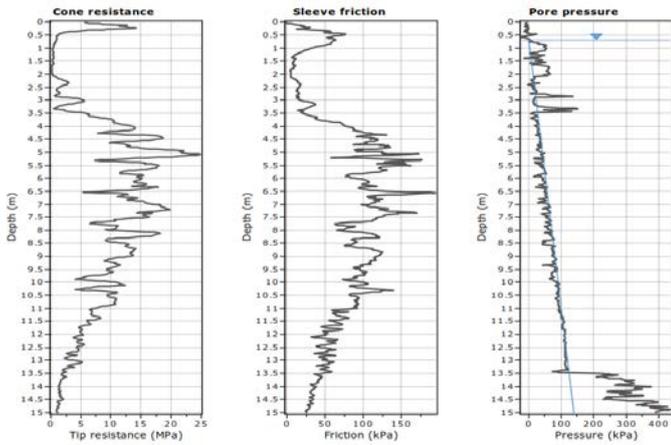


Fig. 5a. CPTU 04 measurements.

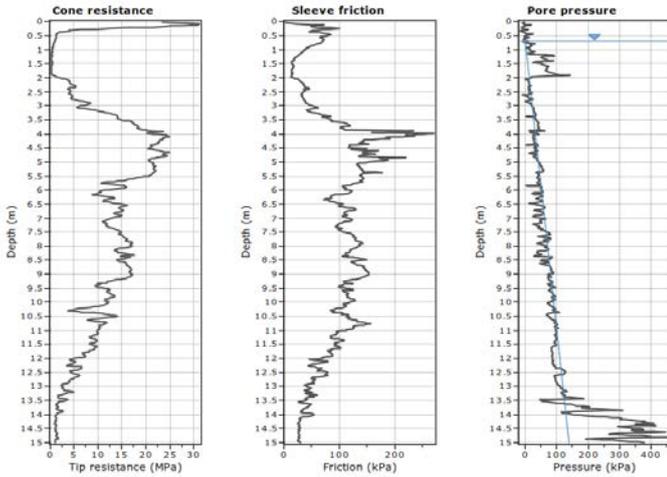


Fig. 5b. CPTU 06 measurements.

2 SDMT tests, SDMT 03 and SDMT 10 are used in this paper to assess the liquefaction potential at the end of the first phase of ground improvement. The results of these tests are presented in Fig. 6a and Fig. 6b.

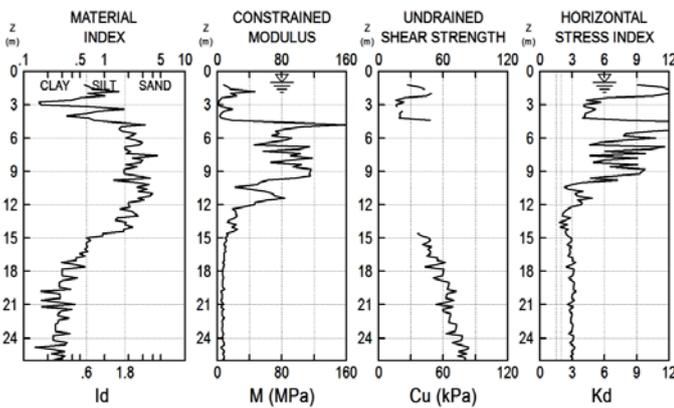


Fig. 6a. SDMT 03 measurements.

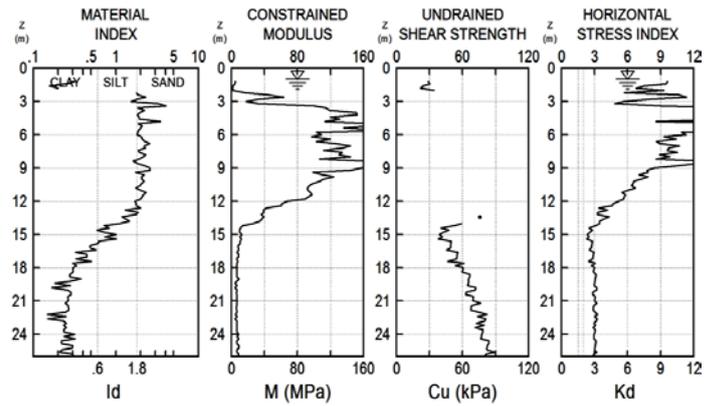


Fig. 6b. SDMT 10 measurements.

### 3 METHODOLOGIES FOR ASSESSING LIQUEFACTION POTENTIAL

#### 3.1 CPTU Based Methods

##### 3.1.1 Overall Approach

The Robertson (2009) and Boulanger & Idriss (2014) methods use a deterministic relation expressed as a factor of safety,  $FoS$ , as the ratio of  $CSR$  to  $CRR$  multiplied to the magnitude scaling factor,  $MSF$ , as given in Eq. (1) below:

$$FoS = \left( \frac{CRR}{CSR} \right) \cdot MSF \quad (1)$$

where,  $FoS$  = factor of safety;  $CRR$  = cyclic resistance ratio,  $CSR$  = seismic induced cyclic stress ratio. If  $FoS > 1$ , no liquefaction is expected.

##### 3.1.2 CSR Calculations

Both methods estimate the  $CSR$  based on the simple approach proposed by Seed & Idriss (1971) given in Eq. (2) below:

$$CSR_{7.5} = \frac{\tau_{av}}{\sigma_{v0}} = 0.65 \cdot \left( \frac{\sigma_{v0} \cdot a_{max}}{\sigma_{v0}} \right) \cdot r_d \quad (2)$$

where,  $\tau_{av}$  = average cyclic shear stress;  $\sigma_{v0}$  = total vertical overburden stress,  $\sigma'_{v0}$  = effective vertical overburden stress,  $a_{max}$  = maximal acceleration of the ground surface in horizontal direction;  $r_d$  = stress reduction factor,  $0.65$  = factor used to convert peak ground cyclic shear stress ratio to a cyclic stress ratio, representative of the most significant cycles over full duration of loading.

The calculation of  $r_d$  differs between Robertson (2009) and Boulanger & Idriss (2014) methods as follows:

- Robertson (2009) method for liquefaction evaluation uses the  $r_d$  value according Liao and

Whitman (1986), which is estimated using depth dependent relationships, given in Eqs. (3) below:

$$r_d = 1 - 0.00765z, \text{ if } z < 9.15 \text{ m} \quad (3a)$$

$$r_d = 1.174 - 0.0267z, \text{ if } 9.15 < z < 23 \text{ m} \quad (3b)$$

$$r_d = 0.744 - 0.008z, \text{ if } 23 < z < 30 \text{ m} \quad (3c)$$

$$r_d = 0.5, \text{ if } z > 30 \text{ m} \quad (3d)$$

- Boulanger & Idriss (2014) method for liquefaction evaluation uses the value of  $r_d$  according to Idris (1999), which is estimated as a function of depth and earthquake magnitude given in Eqs. (4) below:

$$\ln(r_d) = \alpha(z) + \beta(z) \cdot M, \text{ if } z \leq 34 \text{ m} \quad (4a)$$

$$\alpha(z) = -1.012 - 1.126 \cdot \sin\left(\frac{z}{11.73} + 5.133\right) \quad (4b)$$

$$\beta(z) = 0.106 + 0.118 \cdot \sin\left(\frac{z}{11.28} + 5.142\right) \quad (4c)$$

where,  $z$  = the depth below the ground surface.

### 3.1.3 CRR Calculations

Both methods estimate CRR based on cone resistance. Boulanger & Idriss (2014) has been developed for clean sands with  $FC < 5\%$ . If the  $FC > 5\%$ , a correlation factor is engaged and the correction becomes constant if  $FC > 35\%$ . But Robertson (2009) uses the correction factor for fine contents and plasticity index of soils. Both methods imply iterative functions to determine CRR.

The CRR for earthquake magnitude 7.5 according to Robertson (2009) considers the initial stress exponent  $n = 1.0$  and calculates the normalized cone resistance,  $Q_m$ , normalized friction ration,  $F_R$  and soil behaviour type index,  $I_c$ .

The stress exponent is calculated by using Eq. (5) below, which is involved in the calculations of overburden correction factor of penetration,  $C_N$ , as given in Eq. (6) (Robertson 2009)

$$n = 0.38 \cdot I_c + 0.05 \cdot \left(\frac{\sigma'_{v0}}{P_a}\right) - 0.15, n \leq 1.0 \quad (5)$$

$$C_N = \left(\frac{P_a}{\sigma'_{v0}}\right)^n \quad (6)$$

An iteration until the change in  $n$  produces  $\Delta n \leq 1$  is required.

Depending on the values of  $I_c$ , which varies from 1.64 until 2.70, the correction factor for soil

plasticity, fines content, mineralogy, soil sensitivity, age and stress history,  $K_C$  is calculated, which is than involved in the calculations of  $Q_{m,cs}$  and  $CRR_{7.5}$  by using Eq. (7) and (8). (Robertson 2009)

$$Q_{m,cs} = K_C \cdot Q_m \quad (7)$$

$$CRR_{7.5} = 93 \cdot \left(\frac{Q_{m,cs}}{1000}\right)^3 + 0.08, \text{ if } 50 \leq Q_{m,cs} \leq 160 \quad (8)$$

If  $I_c \geq 2.70$ , than CRR is calculated by Eq. (9) below:

$$CRR = 0.053 \cdot Q_m \cdot K_\alpha \quad (9)$$

where,  $K_\alpha$  = correction factor to account to static shear stress.

The values of CRR for earthquake magnitude 7.5 according to Boulanger & Idriss (2014) is calculated also based on the value of cone resistance,  $q_c$ . In this method is also involved in the calculations the  $C_N$ , calculated as follows by Eq. (10):

$$C_N = \left(\frac{P_a}{\sigma'_{v0}}\right)^\beta \leq 1.70 \quad (10)$$

$$\text{where, } \beta = 1.338 - 0.249 \cdot (q_{c1N})^{0.264} \quad (10a)$$

$$\text{where, } q_{c1N} = C_N \cdot q_c$$

At this point, an iteration between  $C_N$  and  $q_{c1N}$  is required. (Boulanger & Idriss 2014)

The derived value of  $FC$  is used to calculate the equivalent clean sand adjustments,  $\Delta q_{c1N}$  by using Eq. (11).

$$\Delta q_{c1N} = (11.9 + \frac{q_{c1N}}{14.6}) \cdot \exp(1.63 - \frac{9.7}{FC + 2} - \left(\frac{15.7}{FC + 2}\right)^2) \quad (11)$$

The equivalent clean sand of CPTU is calculated by using Eq. (12) below:

$$q_{c1N,cs} = q_{c1N} \cdot \Delta q_{c1N} \quad (12)$$

Therefore,  $CRR_{7.5}$  is calculated by Eq. (13) (Boulanger & Idriss 2014).

$$CRR_{7.5} = \exp\left\{ \frac{q_{c1N,cs}}{113} + \left(\frac{q_{c1N,cs}}{1000}\right)^2 - \left[\left(\frac{q_{c1N,cs}}{147}\right)^3 - \left(\frac{q_{c1N,cs}}{137}\right)^4\right] - 2.8 \right\} \quad (13)$$

### 3.1.4 Magnitude Scaling Factor, $MSF$

The magnitude scaling factor,  $MSF$  is used to scale the  $CSR$  estimated for a magnitude of 7.5 for the design earthquake magnitude. According to Boulanger & Idriss,  $MSF$  is calculated by using Eq. (14) (Idriss 1999)

$$MSF = 6.9 \cdot \exp\left(\frac{-M}{4} - 0.058\right) \quad (14)$$

According to Robertson method,  $MSF$  is calculated by using Eq. (15) below (Youd & Idriss 2001):

$$MSF = \frac{1.74}{M^{2.56}} \quad (15)$$

where,  $M$  = the moment magnitude of design earthquake.

$MSF$  and  $r_d$  must be determined by using consistent methodology.

### 3.2 SDMT Based Methods

#### 3.2.1 $K_D$ - $CRR$ correlations

Robertson (2012) proposed the latest correlation between  $K_D$  and  $CRR$ , obtained by replacing  $Q_{cn}$  with  $25K_D$ .  $Q_{cn}$  -  $CRR$  correlations is calculated using Eq. (16):

$$CRR = 93 \cdot \left(0.025 \cdot K_D\right)^3 + 0.08 \quad (16)$$

According to Idriss & Boulanger (2006),  $CRR$  is calculated using Eq. (17) (Marchetti 2014).

$$CRR_{7.5} = \exp\left\{ \frac{\frac{Q_{cN}}{540} + \left(\frac{Q_{cN}}{67}\right)^2 - \left(\frac{Q_{cN}}{80}\right)^3 - \left(\frac{Q_{cN}}{114}\right)^4}{-3} \right\} \quad (17)$$

where,  $Q_{cn} = 25K_D$

#### 3.2.2 $V_s$ - $CRR$ correlations

The relationship  $CRR-V_{SI}$ , for earthquake magnitude  $M = 7.5$ , is approximated by the Eq. 18:

$$CRR_{7.5} = \left[ 0.022 \left( \frac{K_{a1} V_{SI}}{100} \right)^2 + 2.8 \left( \frac{1}{V_{SI}^* - K_{a1} V_{SI}} - \frac{1}{V_{SI}^*} \right) \right] K_{a2} \quad (18)$$

where,  $V_{SI} = V_S (p_a / \sigma'_{vo})^{0.25}$  (18a)

where,  $V_S$  = measured shear wave velocity,  $p_a$  = atmospheric pressure ( $\approx 100$  kPa),  $\sigma'_{vo}$  = initial effective vertical stress in the same units as  $p_a$ ,  $V_{SI}^*$  = limiting upper value of  $V_{SI}$  for liquefaction occurrence, assumed to vary linearly from 200 m/s for soils with fines content of 35 % to 215 m/s for soils with fines content of 5 % or less,  $K_{a1}$  = factor to correct for high  $V_{SI}$  values caused by aging,  $K_{a2}$  = factor to correct for influence of age on  $CRR$ .

### 3.3 $CRR$ based on $Q_{cn}$ and $K_D$

The  $CRR$  is estimated from two one - to - one correlations, from  $Q_{cn}$  or  $K_D$ , or it can be estimated by these two parameters as given in Eq. (19), as an geometric average of these two values of  $CRR$ :

$$AverageCRR = \left[ (CRR_{fromQ_{cn}}) \cdot (CRR_{fromK_D}) \right]^{0.5} \quad (19)$$

## 4 RESULTS OF CALCULATIONS

### 4.1 Results of calculations based on CPTU data

In Fig. 7a and Fig. 7b are presented the results of  $CSR$  and  $CRR$ , which are calculated by using the measurements of CPTU tests in site.

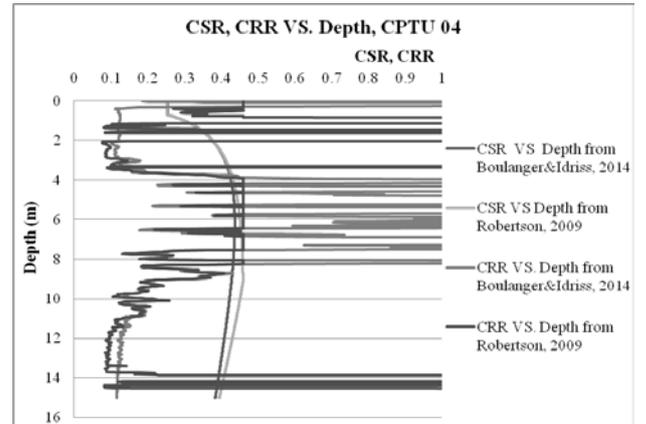


Fig. 7a. CSR and CRR results calculated from CPTU 04.

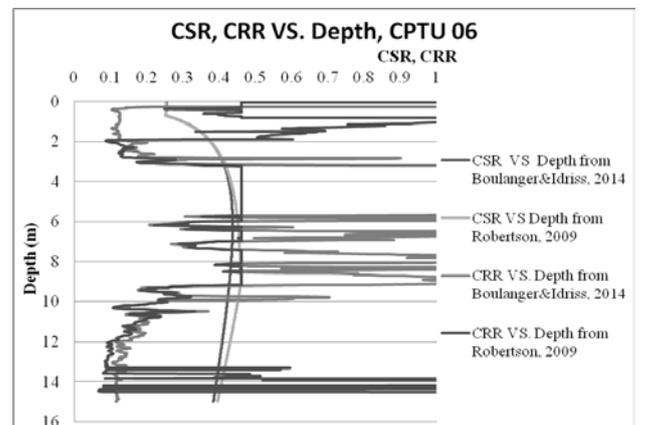


Fig. 7b. CSR and CRR results calculated from CPTU 06.

In Fig. 8a and Fig. 8b are presented the results of the calculated factors of safety,  $FoS$ , by using the measurements of CPTU tests.

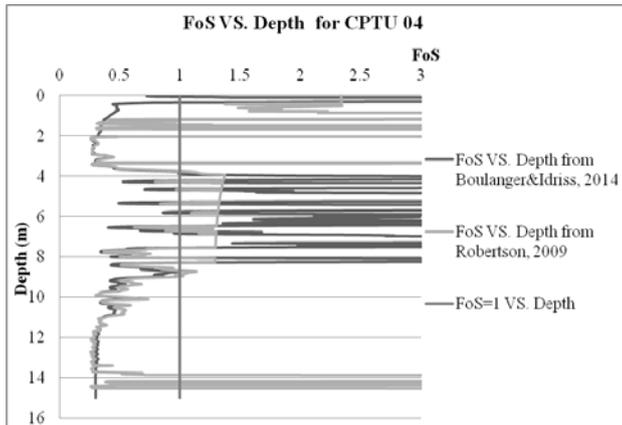


Fig. 8a. FoS results calculated from CPTU 04 data.

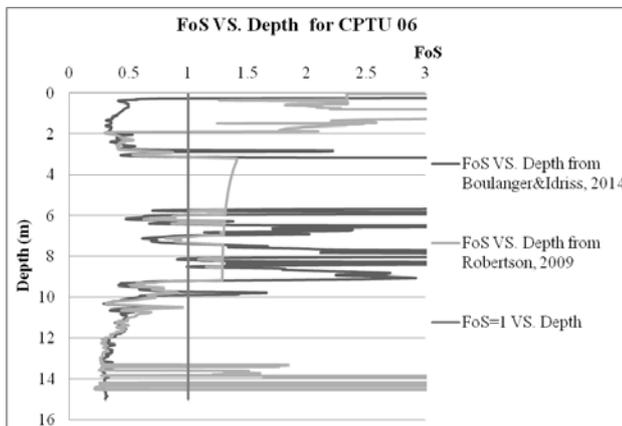


Fig. 8b. FoS results calculated from CPTU 06 data.

#### 4.2 Results of calculations based on SDMT data

In Fig. 9a and Fig. 9b are presented the results of  $CSR$  and  $CRR$ , which are calculated by using the measurements of SDMT tests in site.

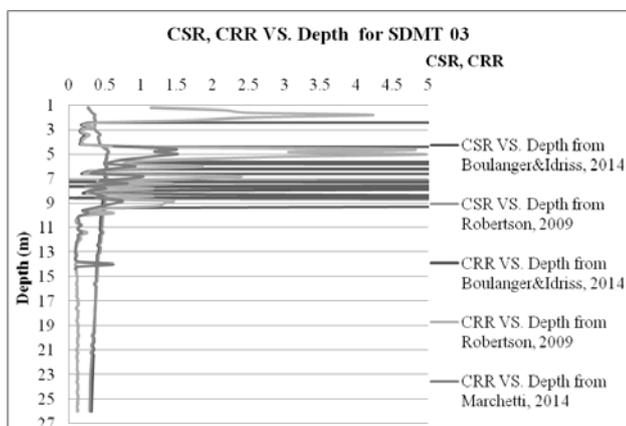


Fig. 9a. CSR and CRR calculated from SDMT 03 data.

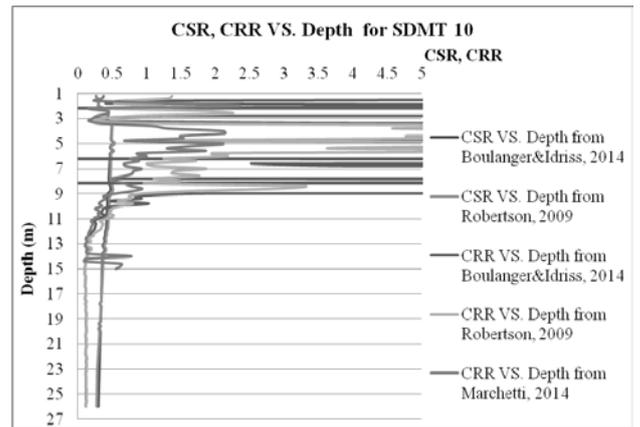


Fig. 9b. CSR and CRR calculated from SDMT 10 data.

In Fig. 10a are presented the results of the calculated factors of safety,  $FoS$ , by using the measurements of SDMT 03 test.

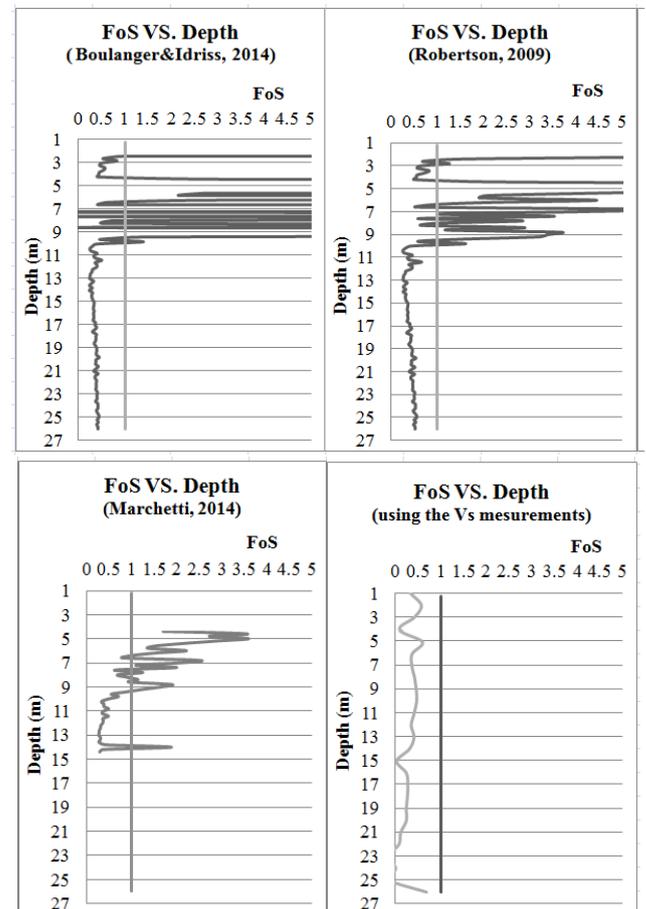


Fig.10a. FoS results calculated from SDMT 03 data.

In Fig. 10b are presented the results of the calculated factors of safety,  $FoS$ , by using the measurements of SDMT 03 test.

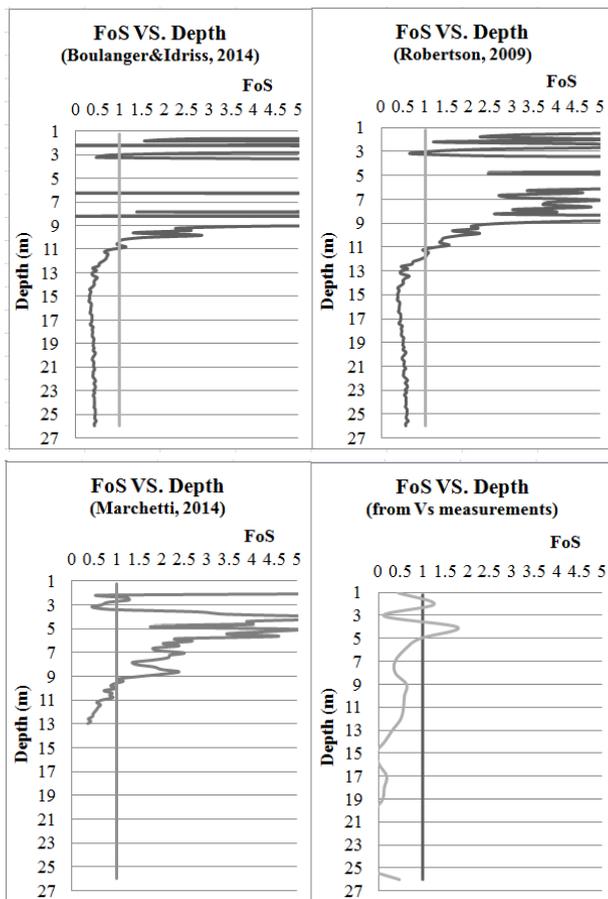


Fig.10b. FoS results calculated from SDMT 10 data.

## 5 CONCLUSIONS

The measurements taken in site by CPTU and SDMT tests are used in this paper to calculate the liquefaction potential. The calculations presented are based on three methods. The results of  $CRR$  based on CPTU measurements are given in Fig. 7a, b. The factors of safety calculated from CPTU measurements are presented in Fig. 8a, b.

According to the calculations based on CPTU 04 and CPTU 06 measurements, which are shown in Fig. 8a, b, from 1 m until a 3 m and from 9 m until 13.5 m, the soils are susceptible to liquefaction.

$CRR$  values calculated based on  $Q_t$  are in good agreement with the  $CRR$  values according to  $K_D$ , as shown in Fig. 9a, b.

According to the results based on SDMT measurements, which are shown in Fig. 10a, b, using the relationship between  $CRR - K_D$ , both methods give the same depths for the layers susceptible to liquefaction, which varies from 2.5 m until 4 m and from 9.5 m until 26 m for SDMT 03 and from 11 m until 26 m, for SDMT 10. These depths are in good agreement for all methods used. Above 11 m the results differ from one method to the other for the calculations based on SDMT 10 data.

The method developed by Marchetti (2014), which is applied only in clean sands, is a good complementary method to the other methods used, because it is based on the data measured by both tests. From CPTU 04 and SDMT 03 measurements it predicts liquefaction from 8.5 m until 13.5 m. From CPTU 06 and SDMT 10 measurements it predicts liquefaction from 2 m until 3 m and from 9 m until 13 m.

Whereas,  $CRR - V_s$  correlations don't provide similar estimations with  $CRR - K_D$  correlation. It appears potentially superior than  $CRR - V_s$ , due to the higher sensitivity of  $K_D$  to relative density  $D_R$  and to other factors that increase liquefaction resistance – above all aging and stress history (Maugeri & Monaco, 2006). Greater weight in this study is given to  $CRR - K_D$  correlations.

Based on these results, it is better to use both CPTU and SDMT parameters to overpass uncertainties and to get a satisfactory reliability in liquefaction potential evaluation.

## 6 REFERENCES

- Boulanger, R. W. and Idriss, I. M. (2014). "CPT and SPT based liquefaction triggering procedures." *Report No. UCD/CGM-14/01, Department of Civil and Environmental Engineering, College of Engineering, University of California at Davis, April, California.*
- Idriss, I.M. & Boulanger, R.W. 2004. "Semi-empirical procedures for evaluating liquefaction potential during earthquakes." *Proc. 11<sup>th</sup> Int. Conf. on Soil Dyn. & Earthquake Engineering & 3<sup>rd</sup> Int. Conf. on Earthquake Geotech. Engineering*, Berkeley, 32-56.
- Japanese Geotechnical Association. (2006). "Remedial measures against soil liquefaction." *Taylor & Francis.*
- Logar, J. (2009). "Geotechnical report: Oil product terminal." *Report no. E-28-09, Ljubjana, Slovenia.*
- Kodi Teknik i Projektimit (KTP-'89). (1989). Albania.
- Marchetti, S. (2014). "Incorporating the stress history parameter  $K_D$  by DMT into the liquefaction correlations." *Personal Communication*, October, Rome, Italy.
- Maugeri, M. and Monaco P. (2006). "Liquefaction potential evaluation by SDMT." *Proc. 2<sup>nd</sup> Int. Conf. on the Flat Dilatometer*, April, Washington DC, 295-305
- Robertson, P. K. and Cabal, K. (2012). "Guide to Cone Penetration Testing for Geotechnical Engineering." *5<sup>th</sup> Edition, Gregg Drilling and Testing, Inc., California.*
- Robertson, P.K. & Wride, C.E. (1998). "Evaluating cyclic liquefaction potential using the cone penetration test." *Canadian Geotech. Jnl*, 35(3), 442-459.
- Seed, H.B. & Idriss, I.M. (1971). "Simplified procedure for evaluating soil liquefaction potential." *Jnl GED, ASCE*, 97(9), 1249-1273.
- Youd, T.L. & Idriss, I.M. (2001). "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils." *Jnl GGE ASCE*, 127(4), 297-313.