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COMPARATIVE SOIL LIQUEFACTION ASSESSMENT BASED ON MULTIPLE INVESTIGATIONS METHODS

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Abstract

Submerged soils like loose sands or very soft clays are sometimes hard to evaluate for geotechnical purposes with classical methods like sampling and lab testing. In such soils, evaluation of liquefaction potential became very important, especially in large projects with hard geotechnical conditions where this issue is involved in almost all stages of design. The paper presents the results of the complex geotechnical evaluation of a site situated on the alluvial plain of the Danube, where simultaneous geotechnical investigations, boreholes with Standard Penetration Test measurements, laboratory analyses, and Cone Penetration Test were completed with geophysical survey (suspension of P-S waves).

Key words

liquefaction potential, safety factor against liquefaction, loose sands

1 Introduction

1.1. Geologic and seismic frame

Romania is a country with an active seismicity, in which more than 300 earthquakes with magnitude M>2.5 (Fig.1- a) occur annually, most of them are superficial and medium magnitude (Bala et al., 2003). The seismicity of Romania is divided into several epicentral areas: Vrancea, Fãgãraş - Câmpulung, Banat, Crişana, Maramureş and Dobrogea. Of these epicentre areas, the Vrancea seismic area is the most important, through the energy of the earthquakes produced, the expansion of their macro-seismicity area and the persistent and concentrated character of the epicenters.

The seismogenic areas bordering the investigated site are Vrancea, Central Dobrogea and Shabla-Cape Kaliakra (Bulgaria).

The Vrancea area represents the most complex seismogenic area being situated at the convergence of at least three tectonic plates: the Eastern European plate, the Moesic Platform and the Intra-Alpine Plate. From the point of view of the depth of the hypocenters, two distinct divisions are distinguished: an area generating normal crustal events up to 40km deep, located between the Peceneaga-Camena fault and the Intra Moesica fault, with moderate magnitude (M_w<5.6), and an area that can generate 3-5 seismic events with M_w>7 per century with intermediate epicentre depths of 70-180km. The strongest earthquake generated here is considered the event of 26.10.1802 (M_w=7.9), followed by the 4 events of the last century 10.11.1940 (M_w=7.7; h=150km), 04.03.1977 (M_w=7.4; h=93km), 30.08.1986 (M_w=7.1; h=131km), respectively 30.05.1990 (M_w=6.9; h=91km) according to Bala et al., (2003) and Vãcãreanu et al., (2016). Almost 90% of seismic mechanisms are reverse fault type with NE-SW oriented planes, which gives the ellipsoidal character of the macroseismic field.

Central Dobrogea seismogenic zone is associated with Capidava-Ovidiu and Horia-Pantelimonu de Sus faults, both of which have transverse character. Between 1980-2010, this area generated 11 events with M_w≥3 and a maximum magnitude M_w=5 (12.12.1986).

The seismogenic zone Shabla - Cape Kaliakra belongs geologically to the Moesian Platform and administratively to Bulgaria, being considered generators of normal crustal earthquakes. In the period 1980-2010, this area generated 15 events with M_w≥4, and the maximum magnitude M_w=7.2 was recorded at the event on 31.03.1901.

From a geological point of view, the studied site conceals a complex structure of the preneogenic foundation and quverture. The deep drilling performed in this county, as geophysical research has revealed the existence of several tectonic compartments, separated by large faults, oriented mostly NW-SE or N-S some limited to the Paleozoic-Mesozoic quilt, others reflecting down to the Neogenic quilt. Fig.1-b, (Oaie et al., 2016) gives the major structural-tectonic architecture of the graben-horst type resulting from the correlation of the above mentioned works, in which we consider that of interest for the front work are the following tectonic elements:

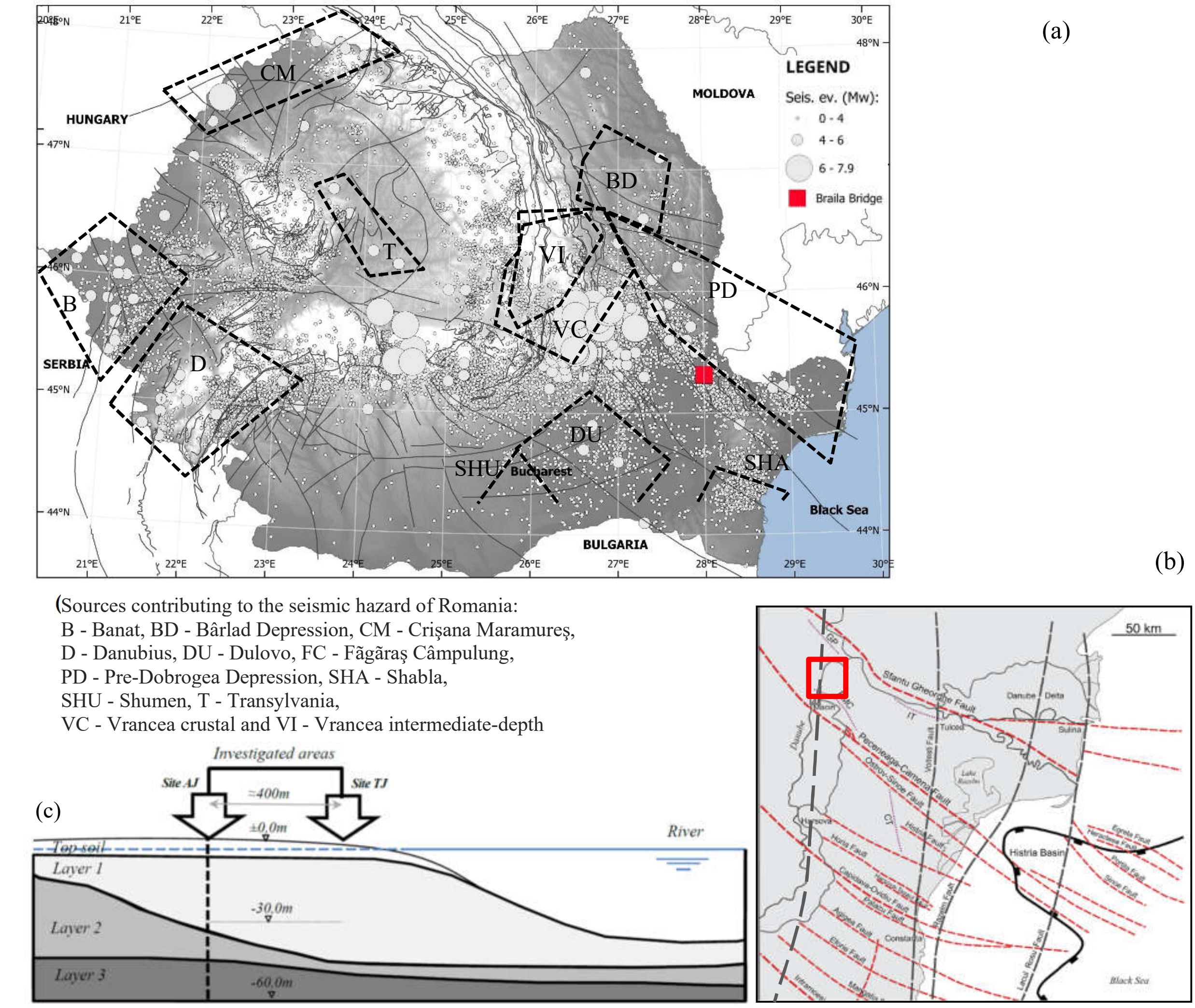

Figure 1. (a) Location of the sites on the seismic map of Romania (compilation after Popa et.al, 2022 and Vãcãreanu et.al., 2016); (b) Tectonic map of Western sector of Black Sea (Oaie et al. , 2016); (c) Sketch of Holocene sedimentary structure of approximate 60m depth.

- the NW-SE directional fault system, seismically active, which is composed of: the Peceneaga-Camena transcrustral fault separating the eastern edge of the Moesic Platform from the North Dobrogean Orogen which is considered active at least in certain segments of the Danube's vicinity, and the profound fault of Saint George which represents the tectonic separation between the North Dobrogea Orogen and the Scythian Platform, being highlighted by numerous gravimetric and magneto-telluric studies, whose work is unanimously accepted and proven by numerous recent earthquakes;
- the NW-SE directional fault system, less active in current times which is composed of: the Voiteşti fault interpreted as a fracture induced by the flexure of the affected deposits, has an N-S direction starting from Tulcea to Kaliakra, it is considered as active in historical times and responsible for barring the intrusion of marine waters into dry territory, and the Danube fault at

the western border of Dobrogea developed in the N-S direction from the northern region of Bulgaria to the area of Fãlciu, on the Prut. It constitutes the deep boundary between the Dobrogea compartment, located at E and the foundation of the Romanian Plain which corresponds to the extension limit of Sarmatian and Pliocene. In the sectors where these regional tectonic accidents intersect, the Intramoesic Fault, The Capidava-Ovidiu Fault, the Peceneaga-Camena Fault and the Saint George Fault, it's recording a higher frequency of seismic events.

The area we refer to is situated at about 90 km distance from the Vrancea and 190km from Shabla seismogenic perimeter. Investigations has been made on two sites, which are located at the flat of Danube River, at 400 m (Site AJ) respectively 0 m distance (Site TJ) from the water course. From engineering perspective, the bedrock is located at more than 300m depth, being covered by shallow deposits consisting in Holocene deposits in fluvial facies which were investigated from surface to a maximum 120m depth. The specific sedimentary structure for this sandy fluvial system consists in (Einsele, 1992): large-scale facies such as floodplain deposits which are composed of small-scale bed types mainly cross-laminated fine sand, silt and mud with frequently peat thin lenses (layer 1), massive, fine sandy silts (layer 2) and gravel bars (layer 3). Figure 1-c contains a sketch of the sedimentary structure described above, with the position of both investigated sites.

1.2. Geotechnical and geophysical investigations

Three types of in situ investigations have been executed on both sites, at distances that do not exceed 50m one from other, which is:

- geotechnical boreholes in continuous rotary dry drilling system, with temporary metal casing protection, sampling system of A class according to ISO 22475-1:2007 and several undisturbed samples in thin Shelby tubes. In all boreholes Standard Penetration Test (*SPT*) has been executed at depth less than 30m.
- cone penetration tests (*CPTu* and *SCPTu*) performed with TE2 equipment types and application class 2 and 3 for *SCPTu* seismic velocities, according to ISO 22476-1:2012;
- geophysical survey (*CH*) for determination of the seismic velocity's profiles along the well depth, respectively "suspension of P-S waves".

The types and number of each investigation executed on both sites are presented in Table 1.

Table 1. Types and number of investigation

Site Investigation		Site AJ	Site TJ
Diameter of the area (m)		76	86
Geotechnical boreholes with SPT tests		4	4
Cone Penetration Tests	CPTu/ SCPTu	4	5
Geophysical survey	P-S	1	1

2 Assessment of liquefaction hazard

Liquefaction is one of the most damaging physical-geological processes associated with the seismic hazard that may damage all granular deposits as the fine ones, of low plasticity. The triggering mechanism depends on many factors that vary in space and time and may be separated into three classes: two related to the soil properties (general settings of soil layers and geomechanical features) and one defined by earthquake characteristics. The prelevation of undisturbed samples in very wet or submerged soils can be an extremely difficult task, sometimes impossible to resolve with usual tools, and thus, in consequence, the assessment of liquefaction hazard by in situ tests became of crucial importance especially in large projects with hard geotechnical conditions, like the one we discuss here.

Presently, assessment of liquefaction susceptibility may be performed in several different ways, (Anwar et al, 2016), either through: (i) probabilistic methods which evaluate the probability of liquefaction (PL), which is a quantitative measure of the severity of this possible phenomenon; (ii) artificial neural networks (ANN) which are conceptual models that estimates the relationships between the earthquake characteristics and the soil with liquefaction potential or, more common, (iii) deterministic methods which provide an alternative verdict of "liquefiable" or "un-liquefiable" based on the computed values of the safety factor against liquefaction (F_{sliq}).

In this work we will approach the third method of research and we will assess the factor of safety against liquefaction (Fs_{liq}) as defined by Ishihara, (1993) and Seed and Harder, (1990):

$$Fs_{liq} = \frac{CRR}{CSR} = \frac{CRR_{7.5}MSF}{CSR} K_\sigma K_\alpha \tag{1}$$

where the significance of terms is: *CRR* - cyclic resistance ratio; $CRR_{7.5}$ - cyclic resistance ratio for an earthquake with 7.5 magnitude; *CSR* - cyclic stress ratio induced by the seismic shake; *MSF* - magnitude scaling factor; K_s - overburden stress correction factor and K_a - correction factor for sloping ground.

The estimation of both *CRR* and *CSR* may be done through numerous semiempirical correlations with in situ test results, some of those are presented in Table 2.

Table 2. List of most used relations involved in the calculation of Fs_{liq}

Parameter	Reference	Parameter	Reference	Site Investigation
CSR	Liao and Whitman,1986	CRR	Seed et al., 1984 and Youd et al., 2001	SPT
	Idriss, 1999		Idriss and Boulanger, 2004	
	Cetin et al, 2004		Japanese Bridge Code-JRA, 1990	
	Youd et al. 2001		Robertson and Wride, 1998	CPT
	Eurocode 8, Part 5		Andrus et al., 2000	CH

2.1. Assessment of CRR

Defined as "the capacity of the soil to resist liqefaction" (Youd et al. 2001), the cyclic resistance ratio may be assessed directly throught cyclic test in the laboratory conditions, or indirectly expresses from correlations of this parameter with the results of standard penetration tests (*SPT*), cone penetration tests (*CPT*) or shear-wave velocity measurements executed in various geophysical tests, such as cross-hole, down-hole, suspension of P-S waves (*CH*) or others. In what follows, due to the underconsolidate and loose state of sediments, very few undisturbed samples has been taken and we had chosen to perform comparative calculations of *CRR* based on *SPT* test, as on *CPT* and *CH* tests, which was executed on both selected sites.

2.1.1. Assessment of CRR based on SPT tests

SPT test is not only the older in situ test, but also the most widespread, so that the blow counts have been put in relation with *CRR* by numerous authors such as those listed in Table 2. The calculations of *CRR* based on *SPT* resistance, was expressed as functions of corrected and normalized values $(N_1)_{60}$ and of clean sand corrected N-value $(N_1)_{60cs}$.

a. In the first set of relations Seed et al., (1984) and Youd et al., (2001) expressed the cyclic resistance ratio for an earthquake with 7.5 magnitude, based on equation (2):

$$CRR_{7.5} = \frac{1}{34-(N_1)_{60cs}} + \frac{(N_1)_{60cs}}{135} + \frac{50}{[10(N_1)_{60cs}+45]^2} - \frac{1}{200} \tag{2}$$

According to Youd et al., (2001), $(N_1)_{60cs}$ may be obtained as a function of fine content (*FC*) from equation (3):

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \tag{3}$$

in which for $FC \leq 5\%$, $a=0$ and $b=1.0$; for $5\% < FC < 35\%$, $\alpha = exp\left[1.76 - \left(\frac{190}{FC^2}\right)\right]$ and $\beta = \left[0.99 + \left(\frac{FC^{1.5}}{1000}\right)\right]$, and finally, for $FC \geq 35\%$ $a=5.0$ and $b=1.2$.

b. In the second set of relations, Idriss and Boulanger, (2004) express $CRR_{7.5}$ from equation (4):

$$CRR_{7.5} = exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right) \quad (4)$$

in which $(N_1)_{60cs}$ has been calculated based on Idriss and Boulanger (2008), formulas (5):

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$$
$$\Delta(N_1)_{60} = exp\left(1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right) \quad (5)$$

c. Finally, Japanese Bridge Code-JRA, (1990) attests that $CRR_{7.5}$ is affected by the median diameter of grain size distribution curve (D_{50}) as follows for 0.05mm $\leq D_{50} \leq$ 0.6mm (6):

$$CRR_{7.5} = 0.0882\sqrt{\frac{(N_1)_{60cs}}{\sigma_v' + 0.7}} + 0.255log\left(\frac{0.35}{D_{50}}\right) + R_3 \quad (6)$$

in which dimensionless coefficient $R_3=0$ for FC<40% and $R_3=0.004FC-0.16$ for $FC \geq 40\%$. We mention that in this third calculation, $(N_1)_{60cs}$ has been used based on Idriss and Boulanger (2004) formulas.

2.1.2. Assessment of CRR based on CPT tests

In international literature the main set of deterministic equations for the evaluation of *CRR* based on *CPT* tests, is given by Robertson and Wride (1998), by the following relations (7, 8).

$$CRR_{7.5} = 0.833\left[\frac{q_{c1Ncs}}{1000}\right]^3 + 0.05, \text{for } q_{c1Ncs} < 50 \quad (7)$$

$$CRR_{7.5} = 93\left[\frac{q_{c1Ncs}}{1000}\right]^3 + 0.08, \text{ for } 50 \leq q_{c1Ncs} < 160 \quad (8)$$

In the above relations, q_{c1N} [-] is the corrected and normalized penetration resistance expressed by (9), q_c [kPa] is the tip penetration resistance, p_a is approximatelly the atmospheric pressure 100 [kPa], σ_{vo}' [kPa] is the effective overburden effort, $(q_{c1N})_{cs}$is the equivalent clean sand normalized penetration resistance (10), K_c is the granulometric correction of CPT (11), I_c is the index of the behavior of the soil (12), expressed on the basis of Q and F (13) and n take values from 0.5 for clean sands to 1 clays.

$$q_{c1N} = \frac{q_c}{\sqrt{\sigma_v' p_a}} \leq 254 \quad (9)$$

$$(q_{c1N})_{cs} = K_c q_{c1N} \quad (10)$$

$$K_c = -0.403I_c^4 + 5.581I_c^3 - 21.63I_c^2 + 33.75I_c - 17.88 \text{ for } I_c>1.64$$
$$K_c = 1, \text{ for } I_c<1.64 \quad (11)$$

$$I_c = [(3.47 - logQ)^2 + (1.22 + logF)^2]^{0.5} \quad (12)$$

$$Q = \left(\frac{q_c - \sigma_{vo}}{p_a}\right)\left(\frac{p_a}{\sigma_{vo}'}\right)^n, F = \left(\frac{f_s}{q_c - \sigma_{vo}}\right)100 \quad (13)$$

2.1.3. Assessment of CRR based on CH tests

Assessement of *CRR* based on shear-wave velocities came into research and practice since 1988, according to Seed and Idriss, (1988), and the most popular method of evaluation is attributed to Andrus and Stroke, (2000). In this method the *CRR* estimation is expressed as follows:

$$CRR = \left[0.022\left(\frac{V_{S1}}{100}\right)^2 + 2.8\left(\frac{1}{(V_{S1}^* - V_{S1})} - \frac{1}{V_{S1}^*}\right)\right]MSF \quad (14)$$

$$V_{S1}^* = V_{S1}\left(\frac{p_a}{\sigma_v'}\right)^{0.25} \quad (15)$$

where Vs is in situ measured shear wave velocity in m/s, and V_{S1}^* is limiting upper value of V_{S1} in m/s, which is theoretically dependent on fine content, (*FC*).

2.2. Assessment of CSR

CSR defined as the average cyclic shear stress induced by shear waves normalized by the initial vertical effective stress (Seed and Idriss 1971), or "the seismic demand on a soil layer" (Youd et al., 2001), is usually expressed using the well-known formula:

$$CSR = 0{,}65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_v}{\sigma_v'}\right)r_d \qquad (16)$$

where the significance of terms is: a_{max} - maximum horizontal ground surface acceleration, g [m/s^2] - gravitational acceleration, s_v [kPa] - total overburden pressure at depth z [m], s'_v [kPa] - effective overburden pressure at depth z and r_d [-] is the stress reduction factor. The latest term, r_d, may be obtain through several analytical methods (Liao and Whitman, 1986; Idriss, 1999; Cetin et al, 2004), but in this application we used the relation of Youd et al. (2001), wich is expressed as follows:

$$\begin{aligned} r_d &= (131 - z)/131, \text{ for } z \leq 9.15\text{m};\\ r_d &= (44 - z)/37, \text{ for } 9{,}15\text{m} \leq z \leq 23\text{m}; \qquad (17)\\ r_d &= (93 - z)/125, \text{ for } 23\text{m} \leq z \leq 30\text{m}; \end{aligned}$$

3. Results

As we presented in paragraph 1.1., both sites have a plane sedimentary unconsolidate structure, for which we performed calculations of Fs_{liq} till 30m depth in the worst scenario of an seismic event of M_w=7.5, in which case *MSF=1* in formula (1). Due to the close position of the locations towards the seimic area Vrancea, for the calculation of CSR we consider the value a_{max}*=0.30g*, according to Romanian Seismic Code. For both sites, the granulosity of sediments consists mainly in sands and silty sands, as presented in Figure 2, with fine contents varying from 1% to 80% and median diameter of granulometric curves from 0.01mm to 0.18mm.

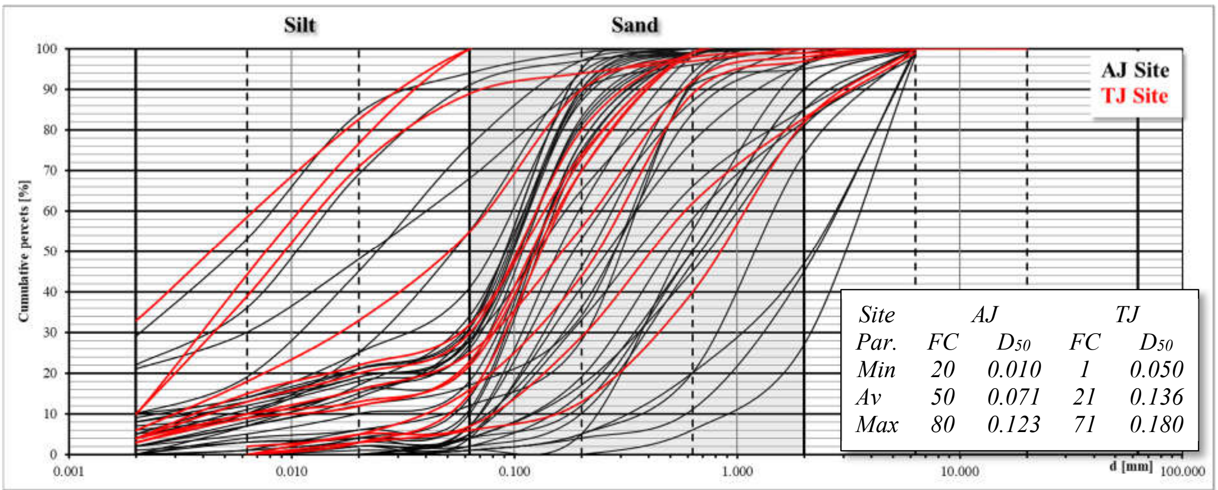

Figure 2. Grainsize distribution curves and specific parameters of investigated sites AJ and TJ.

3.1. Results of the assessment of liquefaction based on SPT tests

In order to estimate the liquefaction hazard based on SPT results we apply all variants of calculation of $CRR_{7.5}$ as described in paragraph 2.1.1. After normalization of *SPT* values, with the aim of calculation of the equivalent clean-sand corrected blow count $(N_1)_{60cs}$, we applied the relations (2÷6), which proved to offer similar results on the first 10m, and slightly to significant different from 10m to 30m depth. For both sites the variation in depth of F_{sliq} (calculated with Seed and Idriss, 1971, *CSR* formula) is presented in Figure 3. We also performed the same calculations using the *CSR* formula provided by Eurocode 8, and results are compared with the previous. Differences in both sites reveals that the second calculation provides larger values of this parameter in the first 10m, and significant smaller below this depth.

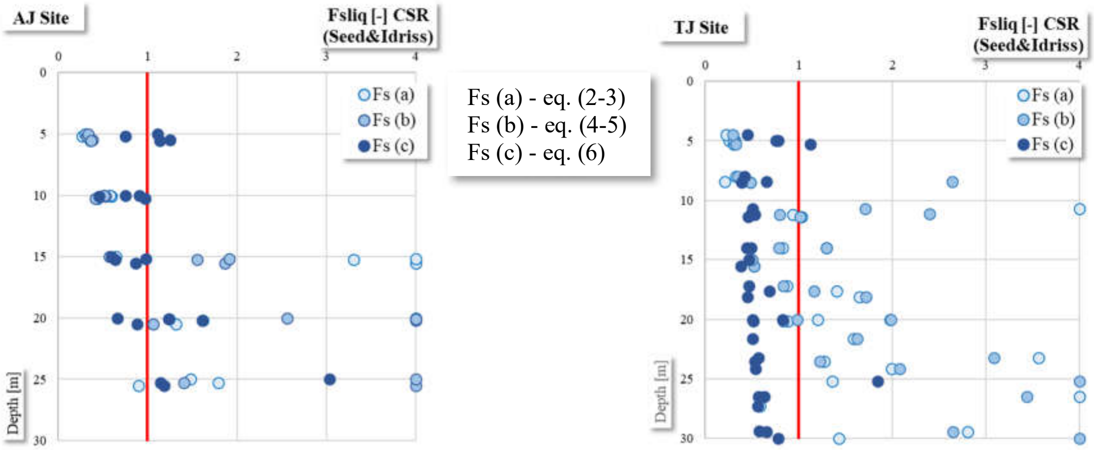

Figure 3. Assessment of factor of safety against liquefaction on AJ and TJ sites, based on *SPT* data.

3.2. Results of the assessment of liquefaction based on CPT tests

The assessment of $CRR_{7.5}$ based on *CPT* results was executed according to the procedure given by Robertson and Wride (1998) in equations 7÷13, for both sites, and the results are graphically represented in Figure 4.

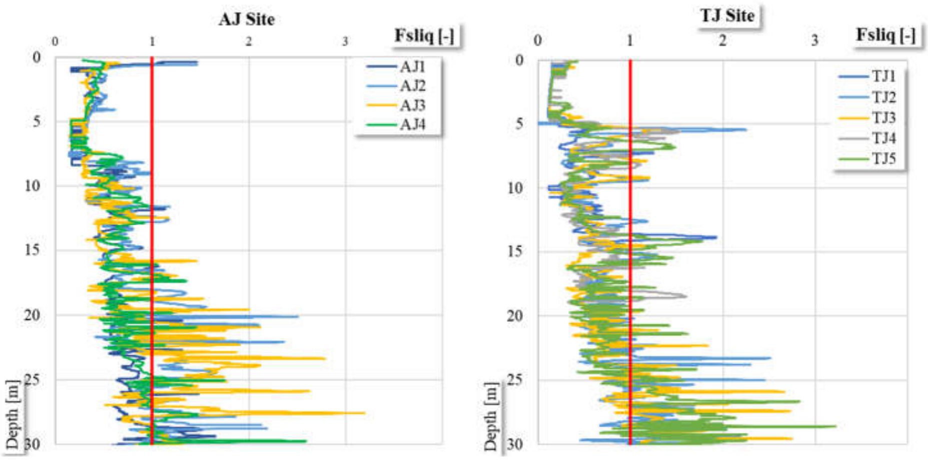

Figure 4. Assessment of factor of safety against liquefaction on AJ and TJ sites, based on *CPT* data

3.3. Results of the assessment of liquefaction based on CH tests

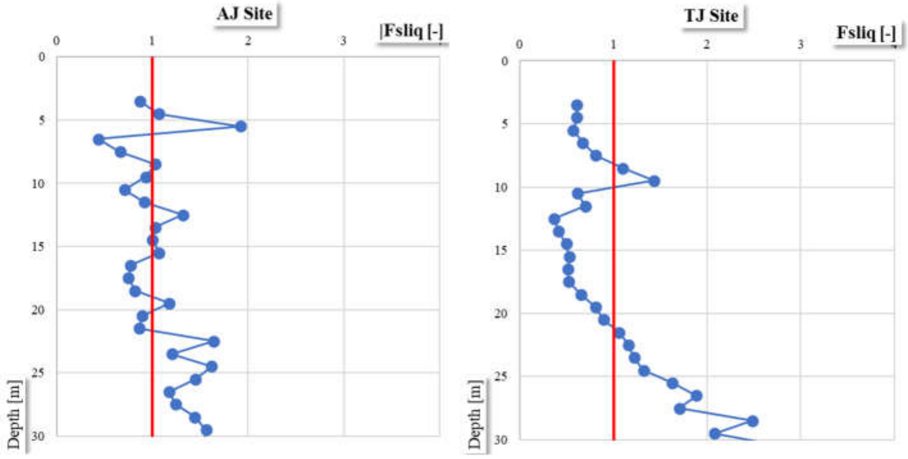

Figure 5. Assesment of factor of safety against liquefaction on A J and TJ sites, based on *CH* data

In the next step, in the purpose of estimation the resistance to liquefaction based on seismic measurements, Andrus and Stroke, (2000) procedure has been applyed on the basis of relations (9) and (10), for both sites, and the results are represented in Figure 5.

3.4. Comparison of results

Finally, after completing the evaluation of liquefiability by the three methods mentioned above, we proceeded to comparison of results. Starting from the reference level obtained from SPT test results (all three formulas), and we found for each site, that even if distances between locations are less than 80-90m, results of various methods do not resemble, except for investigations executed in the same borehole. In the last circumstances, the similitude of the curves of variation Fs_{liq} in depth is better if we relate to equations (4-5), given by Idriss and Boulanger (2004), as it can be visually observed in Figure 6.

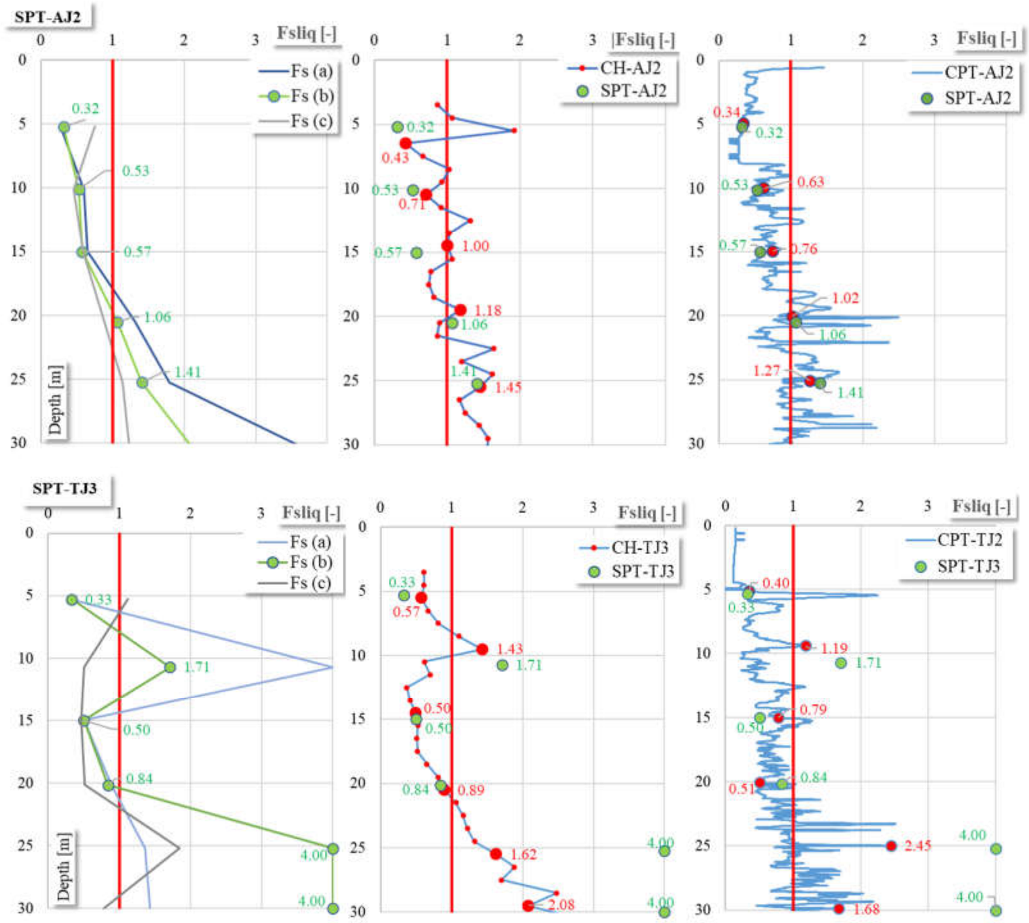

Figure 6. Comparison of safety factor against liquefaction obtained from *SPT*, *CPT* and *CH* tests

Differences obtained in the assessment of *CRR* through the mentioned methods may be explained by the following specific particularities of each method:

- both *SPT* and *CPT* are methods which pushed the soil in large deformations, over the limits of shear failure comparable with those produced by liquefaction, while *CH* measure the resistance of soils submitted to small efforts and deformations, significantly below the limits of equilibrium;
- another significant difference refers to the volume of soil tested: *SPT* and *CPT* are punctual tests which evaluate resistance over a small, limited volume of influence (4÷8 diameters of instruments according to Rogers, 2006), while *CH* test provides average strength properties of larger volumes, which can reach hundreds of meters in diameter depending on the power of the seismic source;
- finally, we conclude that the influence of granulosity over the assessment of *CRR* is variable: (i) in *SPT* assessment is very important, and thus for a prudent and correct application of all three formulas (2÷6), it should be associated with punctual laboratory analyses of grain size distribution of the soil executed at the level of measurement;
  (ii) in the absence of sample prelevation, the influence of granulosity on the values of CRR based on the *CPT* test is more discrete, thanks to the introduction of *Kc* and *Ic* parameters in formulas 7 and 8;
  (iii) in *CH* assessment, the influence of granulosity is implicit, as the measurements of shear waves velocities encompass in an overall mode the mechanical state of soil volume, and are strongly controlled by technical specifications of geophysical sensors.

4. Conclusions

- The comparative analysis of the results of Fs_{liq} calculations based on *SPT*, *CPT* and CH tests, reveals a good agreement only in close adjoining positions of investigations, inspite of the influence of the particularities of each method over results.
- Due to the strong influence of granulosity over the magnitude of *CRR* based on *SPT* test, we conclude that this method must be applyied only if each *SPT* measurements is accompanied by grain size distribution test executed in the same march as the penetration.
- Among the three methods of evaluation of *CRR* based on *SPT* test, those given by Idriss and Boulanger (2004), fits best with the similar calculations based on *CPT* and *CH* tests.
- Finally, due to the fact that all differences exposed above, transposed in terms of Fs_{liq} may conclude in divergent results regarding the liquefaction hazard, the interpretation and comparison of results must be limitted to in situ tests executed simultaneously at minimum distances, or if possible (for geophysical survey), in the same borehole.

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