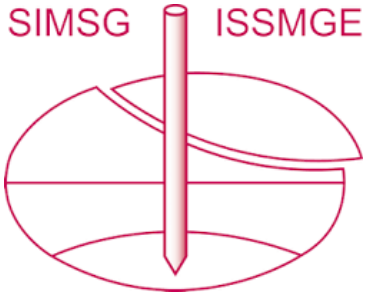
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Comparison of liquefaction susceptibility assessed by various in-situ test methods

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ABSTRACT: The occurrence and extension of soil liquefaction phenomenon is strongly dependent on both seismic excitation and soil characteristics at site under observation. Within this research, the focus was given to a comparison of methods for assessing soil susceptibility to this phenomenon based on field measurements. The resistance of the soil to liquefaction was obtained by different methods requiring in-situ test results, namely from SPT, CPT, DMT and shear wave velocity measurements. The evaluation of cyclic resistance ratio (CRR) was calculated for data captured at three different locations, including vicinity of Hydro Power Plant Brežice site (Slovenia), oil terminal Porto Romano (Albania) and Çanakkale city (Turkey).

Keywords: liquefaction; cyclic resistance ratio; in-situ test

1. Introduction

As a consequence of strong seismic movement in a soil profile containing layers of saturated sandy to silty materials, a soil liquefaction can occur. The extension of this phenomenon is strongly dependent on released seismic energy and soil resistance to instant pore pressure build-up, causing decrease of effective stresses in soil medium and consequent decrease of its shear strength.

Since many in-situ tests are available in geotechnical engineering for ground characterization and evaluation of various engineering parameters, different empirical equations were developed to assess liquefaction susceptibility at site under observation. Within this research, a comparison of some well known in-situ tests, including standard penetration test (SPT), cone penetration test (CPT), dilatometer test (DMT) and measurements of shear wave velocity recorded by different techniques, was performed in terms of cyclic resistance ratio (CRR). CRR values with depth were obtained for three different locations, namely in vicinity of Hydro Power Plant Brežice (Slovenia), oil terminal Porto Romano (Albania) and Çanakkale city (Turkey), where extensive investigation campaigns were carried out. At all selected locations, sandy layers were found in various depths and thicknesses with relatively high underground water level not more than few meters deep due to the vicinity of sea (Porto Romano and Çanakkale) or acumulated river (HPP Brežice). According to the outcomes of SHARE project (Figure 5.) [1], these sites under consideration are located in seismicaly active areas. Moreover, PGA at the surface can be even higher due to the amplification effect through soil profile. High seismicity and the presence of saturated sandy layers indicate high probability of liquefaction occurrence at selected sites.

After brief description of liquefaction phenomenon as well as used empirical equations for the evaluation of CRR, the comparison of results obtained by various procedures and final outcomes are presented below.

2. Methodology

2.1. Liquefaction background

In general, parameters that affect the occurrence and extension of soil liquefaction can be divided into three main categories, namely soil properties, site conditions and seismic parameters [2]. According to the Tang et al. [2], the most influencing factors related to the soil properties are relative density (D_r), grain size distribution, soil permeability, degree of saturation (S_r) and consolidation (OCR). In particular, sandy to silty material are most prone to liquefaction, since large voids do not allow significant build-up of pore pressure in coarser granular soils. Soils with high fines content, as clays, are also less susceptible to liquefaction. However, various criteria have been developed to differentiate between potentially liquefiable and non-liquefiable soils with a significant proportion of fines in combination with index parameters [3-6]. Moreover, it is believed that link between fines content and resistance to liquefaction can be expressed with parabolic relation, where fines content less than critical value act as lubrication between bigger particles or on the contrary when fines content is higher than critical value, they play the role of consolidation and thus increase soil resistance to dynamic excitation [2, 7-9]. In the second group, named as site conditions, thickness and depth of liquefiable layer, ground water table, stratigraphic and morphologic texture, etc. are placed. These factors mainly influence the initial stress state in the soil. With increasing depth of liquefiable layers, effective stresses increase and consequently the shear strength of the soil increases. Soil layer is not likely to liquefy below 20 m depth [2]. Duration of earthquake, magnitude, epicentral distance, frequency component and direction of movements among others are all factors

included into the group of seismic parameters. Liquefaction potential at site under observation is strongly dependant on released energy during earthquake event. The greater the magnitude and the longer the duration of the earthquake, the higher is potential of liquefaction appearance.

Many studies have been conducted using different approaches to evaluate the potential of soil liquefaction, including energy, strain or stress based approach [10-13]. Although the increase of pore water pressure during liquefaction is more related to deformations than stresses in the soil, strain approach is rarely used due to the lack of accurate relations between deformations and in-situ measurements. Within this research the focus was given on the empirical correlations with in-situ tests and past liquefaction observations, based on stress-based approach proposed by Seed and Idriss [13].

2.2. Evaluation of cyclic resistance ratio via different in-situ tests

The main advantage of in-situ investigations is to capture ground information in its actual state. Moreover, wide area can be investigated within a relatively short time and financially acceptable frame. Certain methods, for example CPT, provide a continous record with depth, while with others point measurements (e.g. SPT and DMT) or plain measurements (shear wave velocity profile) are obtained.

Generally, in-situ methods can be divided into two branches, namely, destructive and non-destructive investigations. In addition to the manner in which the investigation is performed, non-destructive and destructive methods differ also in the size of soil deformations at which the measurements are captured. Shear wave velocity measurements (non-destructive) are obtained at small strains, while measurements by SPT, CPT and DMT are performed at medium to large strains. The former are normally performed from the ground surface or between two previously drilled boreholes. Analyzing surface measurements requires a wealth of experience, as we do not have direct contact with the material under investigation. Better interpretations are achived by consideration of additional information about soil medium obtained during drilling or the comparison with results implemented from destructive methods.

Even though various authors have developed different equations for the evaluation of CRR, only one interpretation methodology for each in-situ test was chosen for the comparison presented below. The selection of interpretation methodology was based on literature review and preliminary studies. State of the art equations developed in recent years were used within this research. For the evaluation of CRR from SPT and CPT data, the methodologies developed by the same authors were used, because their equations were developed on similar database of past liquefaction observations and thus uncertainties of comparative analyses may be reduced.

2.2.1. SPT

One of the oldest and frequently used test in geotechnical practice is standard penetration test. Due to the extensive database on a wide range of different soils, there are numerous correlations between field measurements and engineering parameters, including the evaluation of cyclic resistance to soil liquefaction. For the purpose of this research, CRR values were estimated according to the Eq. (1), proposed by Idriss and Boulanger [14]. Furthermore, comparison with Youd et al. curve and case histories of liquefaction observations are shown in Figure 1.

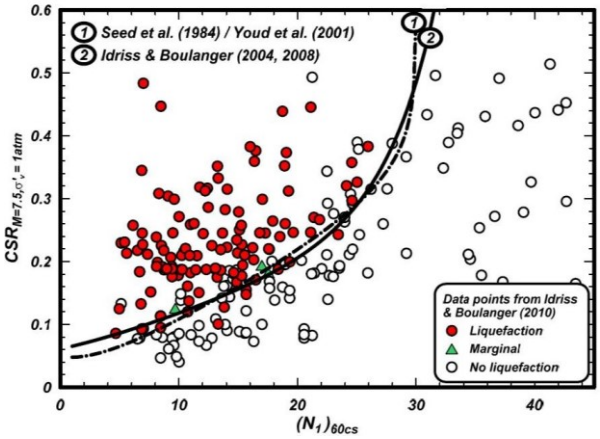


Figure 1: CRR-$(N_1)_{60cs}$ relations by various authors [15].

$$CRR = e^{\left(\frac{(N_1)_{60cs}}{14.1}+\left(\frac{(N_1)_{60cs}}{126}\right)^2-\left(\frac{(N_1)_{60cs}}{23.6}\right)^3+\left(\frac{(N_1)_{60cs}}{25.4}\right)^4-2.8\right)} \quad (1)$$

The CRR in Eq. (1) is computed using the equivalent clean sand corrected blow counts, $(N_1)_{60cs}$. Initially, the field measurements are corrected with correction factors that account for overburden pressure, diameter of borehole, length of the drilling rod, sampler type and hammer energy, while the adjustment due to fines (FC) were computed by Eq. (2) and added to the corrected SPT blows (Eq. (3)) [15].

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

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60} \quad (3)$$

2.2.2. CPT

With the development of sensitive sensors, other devices for ground characterization were developed in the 1950s, including cone penetration test. During the CPT test, sleeve friction, f_s, and tip resistance, q_c, are measured continuously with depth. Robertson and Wride [16] developed the equation using both quantities for the evaluation of CRR, while Boulanger and Idriss [15] take into account only the tip resistance. The latter approah was used within this research (Eq. (4)).

$$CRR = e^{\left(\frac{q_{c1Ncs}}{113}+\left(\frac{q_{c1Ncs}}{1000}\right)^2-\left(\frac{q_{c1Ncs}}{140}\right)^3+\left(\frac{q_{c1Ncs}}{137}\right)^4-2.8\right)} \quad (4)$$

The equivalent clean sand penetration resistance, q_{c1Ncs}, is estimated by an iterative procedure using the following equations (Eqs. (5-9)).

$$q_{c1Ncs} = q_{c1N} + \Delta q_{c1N} \quad (5)$$

$$q_{c1N} = C_N \cdot q_{cN} = C_N \cdot \frac{q_c}{P_\alpha}, \quad (6)$$

$$C_N = \left(\frac{P_\alpha}{\sigma'_v}\right)^m \leq 1.7, \quad (7)$$

$$m = 1.338 - 0.249 \cdot (q_{c1Ncs})^{0.264}, \quad (8)$$

$$\Delta q_{c1N} = \left(11.9 + \frac{q_{c1N}}{14.6}\right) \cdot e^{\left(1.63 - \frac{9.7}{FC+2} - \left(\frac{15.7}{FC+2}\right)^2\right)} \quad (9)$$

According to the authors, the Eq. (4) is limited by the q_{c1Ncs} values between 21 and 254 kPa.

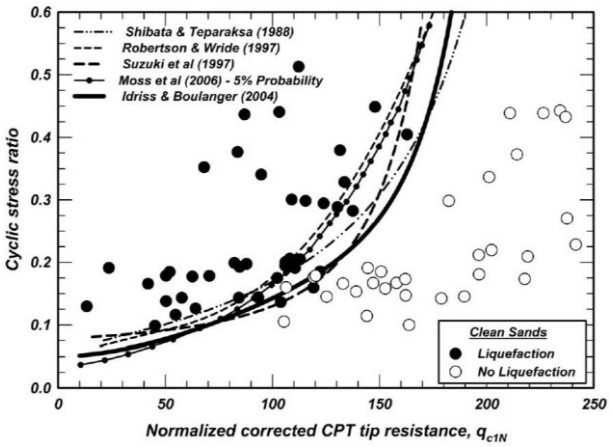


Figure 2: CRR-q_{c1N} relations by various authors [15].

Figure 2 shows comparison of CRR curves calculated on the basis of CPT measurements by different authors.

2.2.3. DMT

Conceptually different soil investigation is performed with flat dilatometer, since the soil is tested in horizontal direction. Furthermore, the penetration of the DMT blade causes less disturbance as compared to SPT and CPT tests and proved to be sensitive to effects of stress history, cementation and ageing via the horizontal stress index K_D. As a consequence it was shown by a number of researchers [17-20] that DMT has great potential for the evaluation of CRR, as well as other engineering parameters. Figure 3 summarizes comparison between some equations for the evaluation of CRR calculated from DMT measurements.

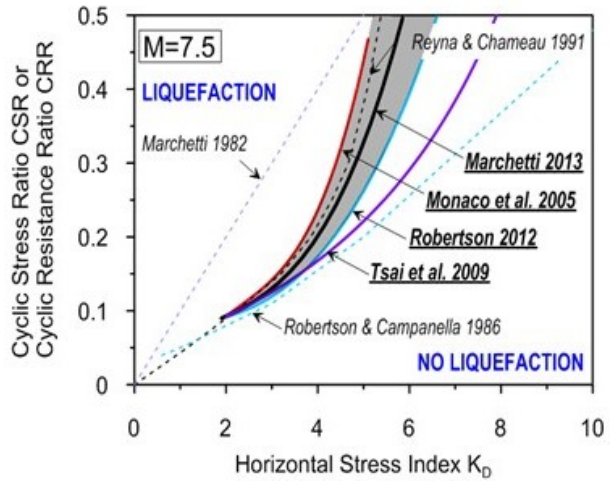


Figure 3: CRR-K_D relations by various authors [21].

Among numerous equations for the calculation of CRR, the one developed by Robertson [22] was selected herein (Eq. (10)).

$$CRR = 93 \cdot (0.025 \cdot K_D)^3 + 0.08 \quad (10)$$

2.2.4. Shear wave velocity

In addition to the above methods, the cyclic resistance ratio can also be estimated from the shear wave velocity profile in the soil. Different techniques are used to measure shear wave velocity, performed from the ground surface or within soil layers (e.g. MASW, seismic refraction, cross-hole, down-hole, up-hole, seismic CPT (SCPT), seismic dilatometer (SDMT)). Regardless of the technique selected, all measurements are performed at small shear strains of the soil (< 0.001 %), where entire nonlinear response of the material can not be captured, and thus raising doubts about the appropriateness of the soil liquefaction resistance assessment via shear wave velocity measurements [23]. Despite the fact that large deformations occur during liquefaction phenomenon, researchers found reasonable correlations between CRR and shear wave velocities. Andrus and Stokoe [24] method was used for the evaluation of CRR using data of shear wave velocities (Eq. (11-13)).

$$CRR = \left(0.022 \cdot \left(\frac{K_{a1} \cdot v_{s1}}{100}\right)^2 + 2.8 \cdot \left(\frac{1}{v_{s1}^* - K_{a1} \cdot v_{s1}} - \frac{1}{v_{s1}^*}\right)\right) \cdot K_{a2} \quad (11)$$

$$v_{s1} = v_s \cdot \left(\frac{P_\alpha}{\sigma'_{v0}}\right)^{0.25} \quad (12)$$

$$v_{s1}^* = \begin{cases} 215 & FC\,(\%) \le 5 \\ 215 - 0.5 \cdot (FC - 5) & 5 < FC(\%) < 35 \\ 200 & FC(\%) \ge 35 \end{cases} \quad (13)$$

The following notation is used in the above equations:

- v_{s1} – normalized shear wave velocity,
- v_s – measured shear wave velocity in m/s,
- $P\alpha$ – atmosferic pressure (101.3 kPa),
- $\sigma_{,v0}$ – vertical effective stress,
- K_{a1} and K_{a2} – correction factors (for uncemented soils of holocene age both factors are 1, otherwise see Figure 4 and Table 1),
- v_{s1}^* – upper limit of normalized shear wave velocity in m/s,
- FC – fines content.

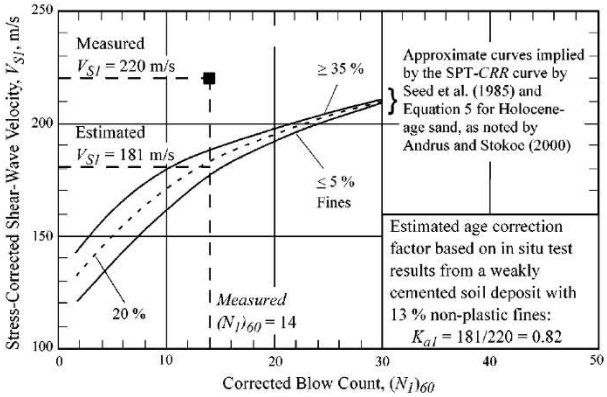


Figure 4: Correction factor K_{a1} [23].

Table 1: Correction factor K_{a2} [23].

Time [Years]	Lower-bound Estimate of K_{a2}
< 10 000	1
10 000	1.1
100 000	1.3
1 000 000	1.5

3. Field measurements (three case studies)

The comparison of some methods for the evaluation of cyclic resistance ratio was made on the data from in-situ measurements performed at three locations, namely in the vicinity of Hydro Power Plant Brežice (Slovenia), oil terminal Porto Romano (Albania) and Çanakkale city (Turkey). All three locations contain deposits of sandy to silty layers, prone to liquefaction concerning the fact that these are seismically active areas with peak ground acceleration (PGA) at bedrock for earthquake with return period 475 years of up to 0.32 g (Brežice), 0.35 g (Porto Romano) and 0.40 g (Çanakkale), respectively [1] (Figure 5).

Extensive investigation campaigns at selected locations are the result of the construction of complex facilities (HPP Brežice and oil terminal Porto Romano) and high population density of the Çanakkale city. The survey inventory of performed in-situ tests and boreholes used within this research are gathered in Table 2.

Table 2: Geotechnical survey at selected locations.

Geotechnical investigation	Brežice	Porto Romano	Çanakkale
Trial pits	6	-	-
Boreholes	7	12	6
SPT	11	33	93
CPT (SCPT)	5	4	2 (4)
DMT (SDMT)	(3)	(2)	3
MASW	1	-	3

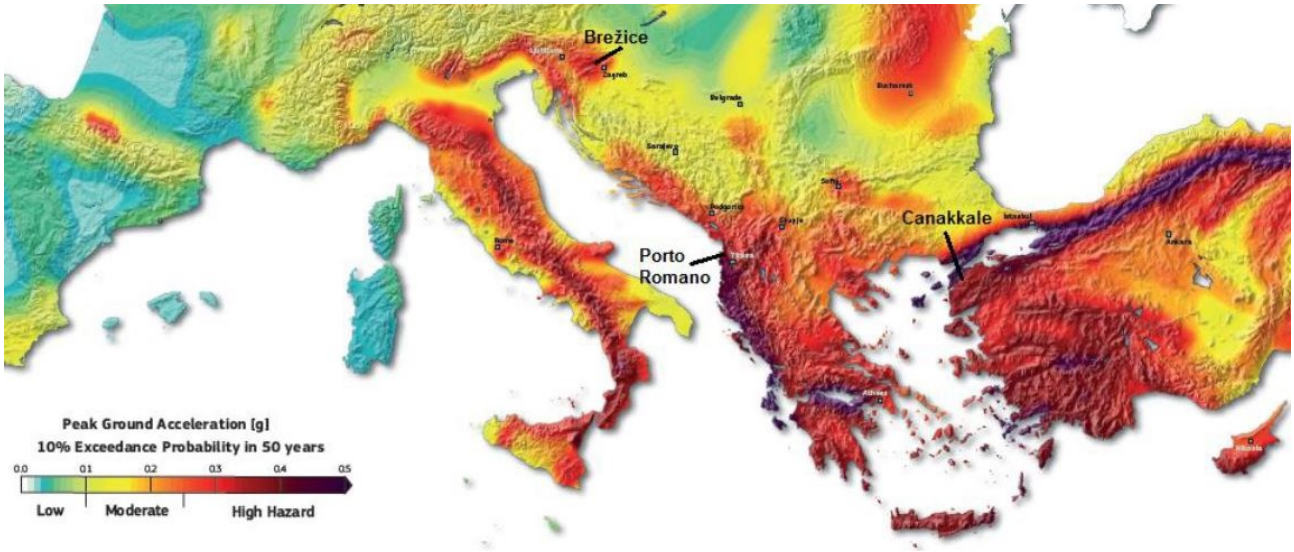


Figure 5: Location of case sites on SHARE earthquake hazard map[1].

The Brežice test field is mainly covered by up to 6 meters thick loose Holocene silty sand layer, deposited over the Quaternary gravel and stiff Miocene marl at the bottom. Water level was predominantly below silty sand layer. With the construction of accumulation basin for the needs of HPP Brežice, the water level in the surrounding area has risen and consequently submerged liquefiable deposits. The area has been considered in the past in terms of potential soil improvement using roller compaction, rapid impact compaction and mixing technique [25 and 26], as well as in terms of liquefaction susceptibility [27].

According to the borehole logs and in-situ tests, around 11.5 m thick liquefiable layer, lying between 1 to 3 m thick clayey crust and Pliocene layers of clay, stretching to the depth of 23 m, were found at test site Porto Romano. Liquefiable layer is very heterogenous in terms of relative density and presence of very thin non-liquefiable clay-like sublayers, which can be deduced from in-situ measurements and consequently from CRR evaluation (see Figure 9).

Unlike Brežice and Porto Romano case sites, where presented geotechnical investigations have been performed at close distance, in-situ tests for Çanakkale case study were performed at six subareas around the city. In general, the whole area is dominated by sands up to 25 m thick with intermediate thin clayey layers. Beneath liquefiable deposits a bedrock of different rock formations (marl, limestone or sandstone) is placed.

Example of SDMT measurements at Brežice test site is presented in Figure 6.

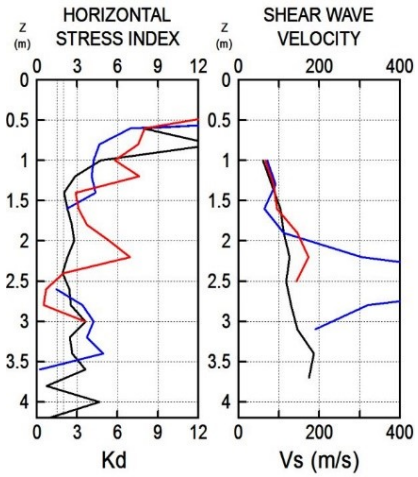


Figure 6: SDMT measurements (Brežice).

4. Results

Final comparison of the CRR values with depth, calculated on the basis of different in-situ measurements for each test site are collected in Figure 8 to Figure 11. Although more tests were performed at three sites, only those obtained at the same microlocation of each site are compared to each other and presented below. Moreover, Figure 7 summarizes the content of fines with depth for presented microlocations, since it considerably affects the soil`s resistance to liquefaction. Fines content was estimated based on CPT measurements (Porto Romano) in combination with additional index and classification tests in the laboratory (Brežice and Çanakkale).

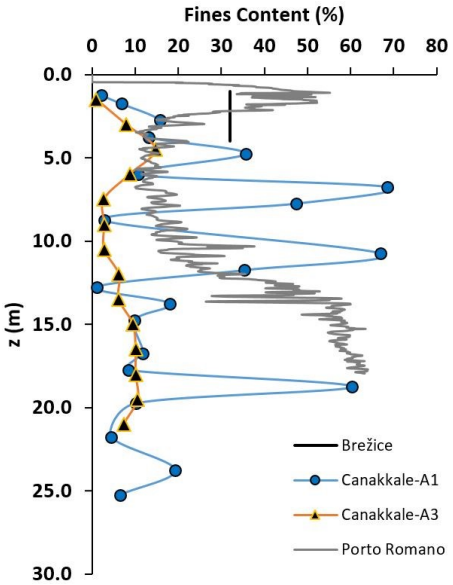


Figure 7: Fines content with depth at test site microlocations.

The results of CRR are presented only for liquefiable layers. Other layers (crust, clay, gravel layers) are shaded in gray in the figures.

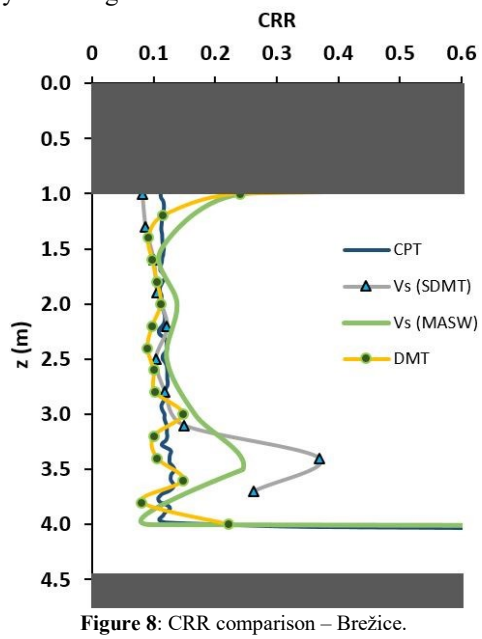


Figure 8: CRR comparison – Brežice.

Figure 8 summarizes calculations of CRR values with depth for test site Brežice. According to the results, CRR values are comparable with each other, indicating a homogenous soil profile of loose uncemented deposits. Only at depths between 3.2 m and 3.7 m, CRR values obtained from shear wave velocities exhibit greater soil resistance.

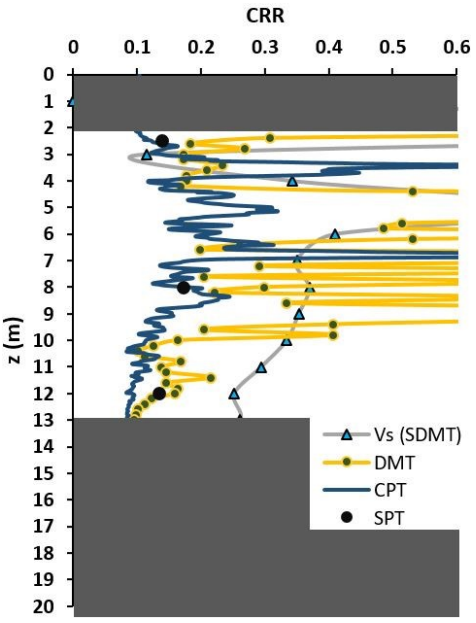


Figure 9: CRR comparison – Porto Romano.

In Figure 9, cyclic resistance at Porto Romano case study is shown. A zigzag pattern of CRR curves between depth of 2 m and 10 m indicate a presence of thin non-liquefiable layers within sandy material, and rapid changes in relative density of sandy material. At the same depth range CRR values obtained from CPT and SPT measurements match well, while higher values were estimated on the basis of DMT and shear wave velocity measurements. Nonetheless, minimum values of CRR obtained from DMT test coincide with CPT based CRR values. The source of some difference may also be the sampling method itself in heterogeneous soils due to the different position of measuring device (horizontal direction of measurements by the DMT test and vertical in the CPT test). Beneath the depth of 10.5 m, sandy to silty material is more homogenous and thus the difference in CRR reduces.

Estimations of CRR for two subareas at Çanakkale test site are gathered in Figure 10 and Figure 11. As for the Porto Romano site, higher CRR values were obtained from DMT test at subarea A1 at Çanakkale site (Figure 10). On the other hand, all other methods coincide fairly well at both presented subareas, with the exception of CRR estimation based on SPT blow counts. It is belived that the difference occur due to poor SPT measurements (questionable equipment status, calibration procedure, human factor, etc.). In the upper 4 to 5 m, the values of cyclic resistance ratio obtained through DMT and shear wave velocity based methods exhibit higher resilience as a result of overconsolidation, high lateral stresses and relative density of the material due to urban environment at the surface. However, CPT based estimation of CRR, gives the lowest values at Çanakkale test site.

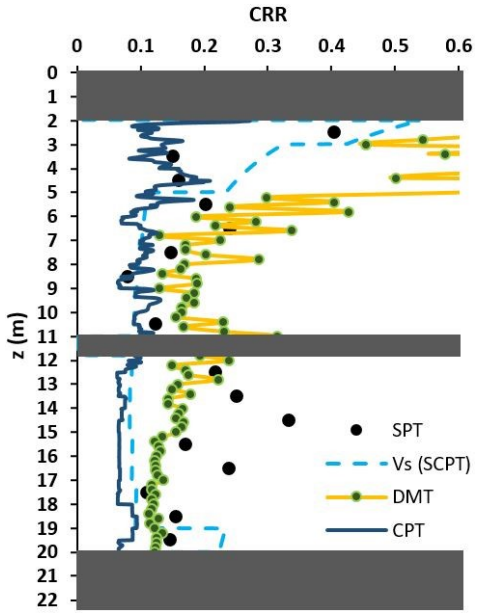


Figure 10: CRR comparison –Çanakkale (subarea A1).

In the analyzes, the minumum shear wave velocities (120 to 190 m/s) obtained through various measurements (SDMT, SCPT, MASW …) were used, in order to obtain the safest results of more sensitive method. The measuring technique used in the CRR comparisons is labelled in the legend in brackets.

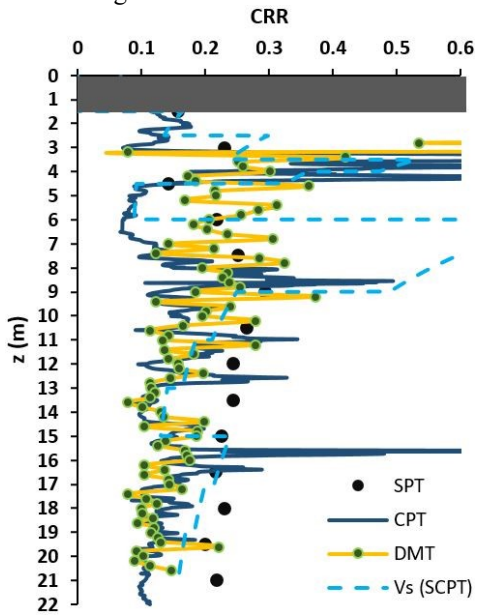


Figure 11: CRR comparison –Çanakkale (subarea A3).

5. Conclusion

Soil resistance to liquefaction phenomenon was estimated in terms of cyclic resistance ratio obtained through four in-situ tests, including SPT, CPT, DMT and shear wave velocity measurements. In-situ tests were performed at three locations, namely near vicinity of HPP Brežice in SE part of Slovenia, oil terminal Porto Romano at west coast of Albania and around Çanakkale city, laying on the Asian part of Dardanelles strait. Although the literature offers many equations for the calculation of the cyclic resistance ratio, only one has been used for each in-situ method, Idriss and Boulanger`s for SPT [14], Boulanger and Idriss`s for CPT [15], Robertson`s for DMT [22], and Andrus and Stokoe`s for shear wave velocity measurements [24].

According to the obtained results the following findings might be exposed:

- Compared to other used methods, the lowest CRR values have been obtained by CPT measurements,
- the evaluation of SPT based CRR values depends on carefull documentation of all influencing parameters that are reflected by correction factors,
- higher resistance to liquefaction estimated from the DMT measurements may suggest the presence of overconsolidated soils or effects of cementation and other factors, since K_D parameter is sensitive to stress history. In addition, with horizontally executed measurements by DMT test a thin rigid layer might be detected, that could be overlooked or averaged within testing zone by the implementation of CPT and SPT tests,
- the evaluation of CRR values from DMT measurements are less reliable for soils with high fines content, since no correction factors are included in empirical equations,
- conducting SCPT and SDMT tests enables the assessment of CRR through two physically different quantities (q_c or K_D and shear wave velocity) obtained with simultaneous probing,
- methods using shear wave velocity data are very sensitive, especially at velocities equal or greater than 200 m/s. Small changes cause significant reduction/raise of CRR,
- in general, the larger the set of methods and the availability of data from the same location, the better the estimation of cyclic resistance to soil liquefaction for further needs. In case of demanding analyses, it is recommended to combine in-situ methods with laboratory results.

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