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Liquefaction potential of sands at the Krško-Brežice field, Slovenia

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Abstract

The Krško-Brežice field is one of the most seismically active areas in Slovenia. The most damaging recorded earthquake with an intensity of VIII (EMS) occurred on 29th January 1917. It caused damage and claimed two lives. In the last 100 years, 9 earthquakes with intensity higher than VI (EMS) have been recorded.

At the investigated area, a top layer up to 5 m thick, consisting of recent deposit of very loose silts and sands (ML, SM, SP), covers the medium dense to dense Quaternary gravel, beneath which there are over-consolidated, uncemented Miocene silts and marls. The top layer could be prone to liquefaction, as reported for the close surroundings of Brežice, where the liquefaction phenomenon was observed during the Zagreb earthquake in 1880 and during the Kupa Valley earthquake in 1909.

The paper presents the results of laboratory index tests, cyclic simple shear tests and field investigations (SPT, CPT, (S)DMT, v_s measurements), which were carried out to assess the liquefaction potential of the top layer at the location of the Brežice Hydroelectric Power Plant (HPP). All results show that the top layer is prone to liquefaction for an earthquake with a 475 year return period. Cyclic simple shear test results show that the liquefaction potential of horizontal ground for an earthquake with a 475 year return period can be reduced by the densification of the top layer to at least 95% of maximum Proctor density.

Keywords: earthquake, CSR, liquefaction potential, silty sand, laboratory investigations, field tests

1. INTRODUCTION

Earthquake induced liquefaction is the phenomenon in which loose saturated granular soil loses shear strength due to pore pressure increase, to the point where it is unable to support structures and to remain stable. Its devastating effects came to the attention of geotechnical engineers in 1964 when the Good Friday earthquake (Magnitude, M = 9.2) in Alaska was followed by the Niigata earthquake (M = 7.5) in Japan (KRAMER, 1996). The phenomenon of liquefaction (including sand volcanoes) was observed also in close surroundings of the Krško-Brežice field, in comparable geological conditions, during the Zagreb earthquake of 1880, in the villages located in the valley of the Sava River, and during the Kupa Valley earthquake of 1909 (VEINOVIĆ et al., 2007, HERAK et al., 2009; HERAK & HERAK, 2010). Recently, the liquefaction in Christchurch, New Zealand (2011) and in Indonesia (2018) reminded other professionals and scientists of its devastating effects.

Until the HAICHENG (1975) and TANGSHAN (1976) earthquakes it was believed that only clean sands were prone to liquefaction (SHENGCONG & TATSUOKA, 1984). Recent investigations have highlighted that earthquakes can trigger shear strength loss in a broad range of types of saturated soils, from sand to low plasticity clays (BRAY et al., 2004, CHU et al., 2004, BOULANGER & IDRISS, 2006), although earthquake-induced ground failure is observed less frequently in clays than in sands. CUBRINOVSKI et al. (2018) reported liquefaction in well-graded gravel with at least 30% of sands and silts during the Kaikoura (New Zealand) earthquake (2016).

Factors governing liquefaction may be grouped into a geological profile, consisting of seismicity at the location (e.g. peak ground acceleration, a_{max}, magnitude, M) and soil properties (e.g. liquid limit, w_L, plasticity index, I_P, density, ρ, grain size distribution, number of cycles to liquefaction, N, cyclic resistance ratio, CRR). The liquefaction potential of soils can be assessed from different laboratory and field investigations by several liquefaction susceptibility criteria, mostly developed for sands (POLITO & MARTIN, 2001, YOUD et al., 2001, ANDRUS et al., 2004, BOULANGER & IDRISS, 2016, 2006, IDRISS & BOULANGER, 2006, MARCHETTI & MARCHETTI, 2016, among others).

This paper presents the assessment of liquefaction potential of the top layer at the location of the Brežice HPP, based on laboratory index and cyclic simple shear tests, as well as field cone penetration tests (CPT/CPTu), standard penetration tests (SPT), flat (seismic) dilatometer tests ((S)DMT) and shear wave velocity measurements (v_s).

2. STUDY AREA

2.1. Geological and hydrological setting

The top layer at the location (Fig. 1) is up to 5 m thick and consists of loose alluvial silty sands and contains fine coal particles that originate from the flotation processes from the area around 60 km upstream, where the coal mines have operated for the last 300 years. Poplar tree roots appear up to 4 m depth and prove the development of the "palaeo" ground during large flood events (PETKOVŠEK et al., 2017).

Beneath the top layer, the medium dense to dense Quaternary gravels classified as silty sandy gravel (GM) to poorly graded gravel (GP) are deposited in thicknesses of between

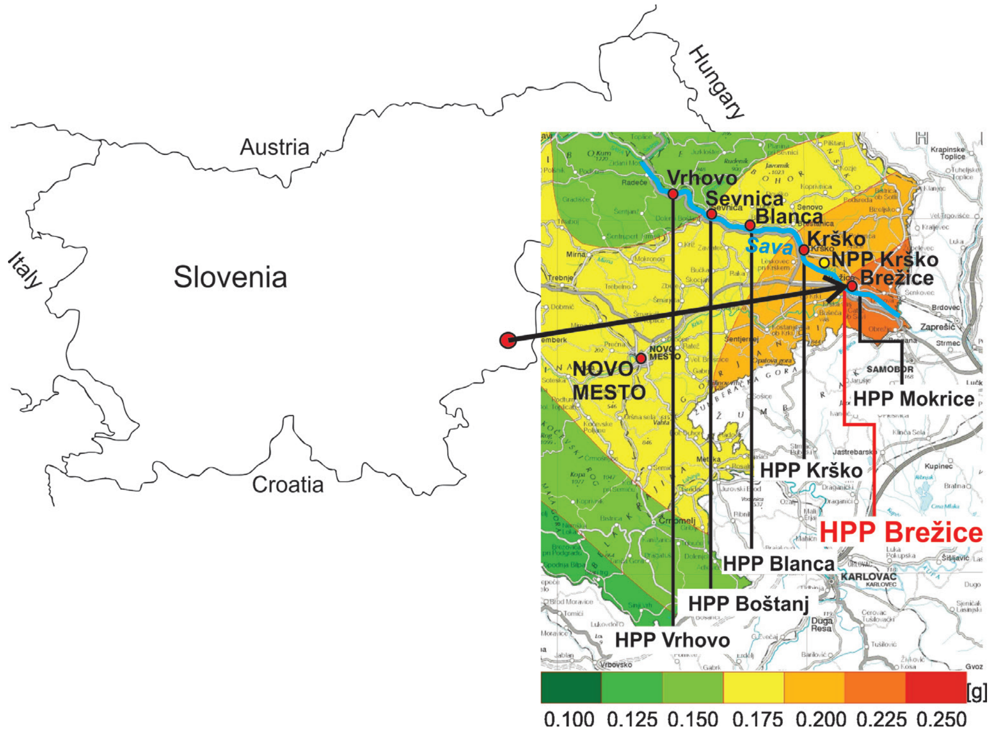

Figure 1. Location of the Brežice HPP – the investigated area in the chain of the lower Sava river Hydroelectric Power plants and the Krško Nuclear Power Plant (NPP). Different colours represent design ground accelerations in rock or firm soil (ARSO, 2001).

8 and 12 m. The bedrock consists of over-consolidated, uncemented Miocene silt and marl, with some inclusions of weak, soft limestone.

The ground water level ranges from 4 - 6 m below the ground surface and follows the water level fluctuation in the Sava River, which frequently floods the area. During floods, the water level

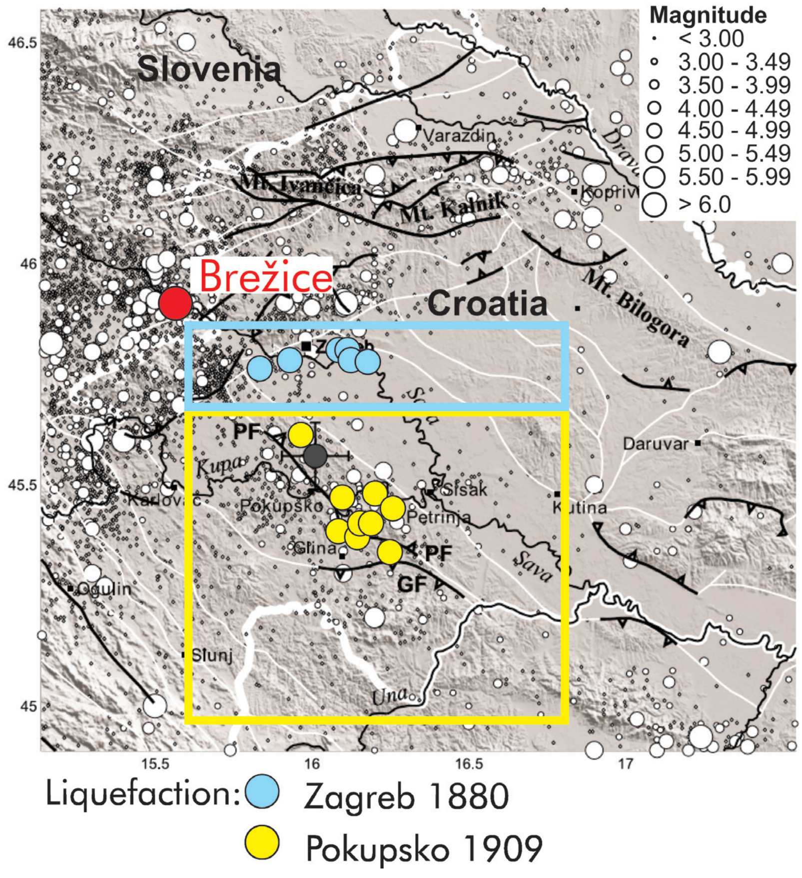

Figure 2. Epicentres of earthquakes at the Krško-Brežice field and in its close surroundings from 567 to 2007 AD, and the locations where liquefaction-related effects were observed during the Zagreb (blue dots) and Kupa Valley (yellow dots) earthquakes (VEINOVIĆ et al., 2007, HERAK et al., 2009, HERAK & HERAK, 2010).

Table 1. Design ground accelerations at the location of the Brežice HPP.

method	return period	magnitude	design ground acceleration	
			rock or firm soil	loose silt, sand
Slovenian national probabilistic seismic hazard analysis (ARSO, 2001)	475	-	0.225 *g*	-
	1000	-	0.250 *g*	-
detailed probabilistic seismic hazard analysis (ALEKSOVSKI et al., 2008)	200	5.6	0.230 *g*	0.290 *g*
	475	5.8	0.280 *g*	0.350 *g*
	1000	6.25	0.295 *g*	0.370 *g*
SHARE model (LAI et al., 2017)	475	-	0.320 *g*	-

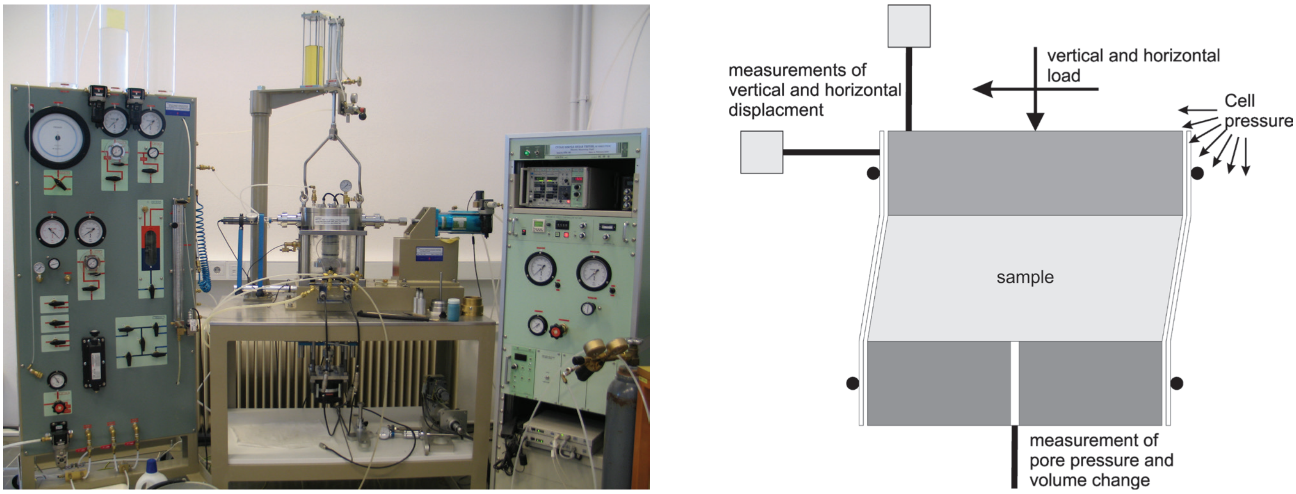

Figure 3. Cyclic simple shear apparatus (left) and schematic illustration of the investigation (right).

rises above the ground surface and may remain for days (PETKOVŠEK et al., 2017). Inside the influential area of the Brežice HPP, the groundwater regime has been changed due to the operation of the HPP Brežice.

2.2. Seismicity

Figure 2 shows the epicentres and magnitudes of earthquakes at the Krško-Brežice field and in its close surroundings from 567 to 2007 AD. Blue and yellow symbols represent locations in Croatia where liquefaction was observed during the Zagreb (1880) and Kupa Valley (1909) earthquakes (after VEINOVIĆ et al., 2007 and HERAK & HERAK, 2010).

The design ground acceleration at the location (Table 1) was predicted based on three different methodologies: (1) currently valid Slovenian national probabilistic seismic hazard analysis (Fig. 1) (ARSO, 2001), (2) detailed probabilistic seismic hazard analysis at the location of the HPP Brežice (ALEKSOVSKI et al., 2008) and (3) recent Seismic hazard harmonization in Europe - SHARE model (LAI et al., 2017).

2.3. Earthquake induced Cyclic Stress Ratio (CSR)

CSR can be estimated using eq. 1, developed as part of the Simplified Liquefaction Procedure (BOULANGER & IDRISS, 2014):

$$CSR_{M,\sigma_v'} = 0.65 \cdot \frac{\sigma_v}{\sigma_v'} \cdot \frac{a_{max}}{g} \cdot r_d \qquad (1)$$

where $a_{\max}$ is the peak ground surface acceleration, g the acceleration of gravity, σ_v the total vertical stress at depth z, σ_v' the effective vertical stress at depth z and r_d shear stress reduction factor that accounts for the dynamic response of the soil profile.

Based on IDRISS (1999), parameter r_d could be calculated using eq. 2.

$$r_d = \exp\left[\alpha(z) + \beta(z) \cdot M\right] \qquad (2)$$

$$\alpha(z) = -1.012 - 1.126 \cdot sin\left(\frac{z}{11.73} + 5.133\right) \qquad (3)$$

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

where z (m) is the depth below the ground surface, M moment magnitude and the arguments inside the *sin* terms are in radians.

The case history CSR values adjusted to a reference magnitude $M = 7.5$ and $\sigma_v' = 1$ atm = 101 kPa could be calculated using eq. 5 and eq. 6 (BOULANGER & IDRISS, 2014).

$$CSR_{M=7.5,\sigma_v'=1} = \frac{CSR_{M,\sigma_v'}}{MSF \cdot K_\sigma} \qquad (5)$$

$$MSF = 6.9 \cdot exp\left(\frac{-M}{4}\right) - 0.058 \leq 1.8 \qquad (6)$$

where $K_\sigma \approx 1$.

$CSR_{M=7.5,\sigma_v'=1}$ varies with depth. In this study it is evaluated for depths between 0.5 m and 4.0 m, for an earthquake with a 475 year return period, design ground acceleration of 0.350 g (Table 1, loose silt, sand) and GW level of 0.1 m. Using parameters listed above, $CSR_{M=7.5,\sigma_v'=1}$ is between 0.30 and 0.35.

3. EXPERIMENTAL METHODS

Preliminary assessment of the top layer liquefaction potential was carried out based on its grain size distribution (SIST-TS CEN

Table 2. Index properties of three samples from the top layer (adapted after PETKOVŠEK et al., 2017).

Parameter	Test Method	sample 1	sample 2	sample 3
USCS Classification	ASTM D2487-10	ML/CL	SC	SM
Natural water content, w_0 (%)	SIST-TS CEN ISO/TS 17892-1	5.06	16.3	21.5
Natural density, ρ (t/m^3)	SIST-TS CEN ISO/TS 17892-2	1.30	1.28	1.23
Dry density, ρ_d (t/m^3)	SIST-TS CEN ISO/TS 17892-2	1.23	1.10	1.01
Void ratio, natural state, e_0 (-)	–	1.15	1.53	1.61
Particle density, ρ_s (t/m^3)	SIST-TS CEN ISO/TS 17892-3	2.64	2.78	2.64
Fines content, < 0.063 mm (%)	SIST-TS CEN ISO/TS 17892-4	54	38	19
Liquid limit, w_L (%)	SIST-TS CEN ISO/TS 17892-12	30	24–26	39
Plasticity index, I_P (%)	SIST-TS CEN ISO/TS 17892-12	9	6–7	–
Optimum water content, SPCT*, w_{opt} (%)	DIN 18127	17.4	15.0	27.5
Maximum dry density, SPCT*, ρ_{dmax} (t/m^3)	DIN 18127	1.65	1.77	1.34

*SPCT – Standard Proctor compaction test

ISO/TS 17892-4) and Atterberg limits (SIST-TS CEN ISO/TS 17892-12).

The laboratory cyclic simple shear apparatus (Fig. 3, left), developed by Seiken Inc., was used to study the liquefaction potential of the top layer at natural void ratio and to evaluate the densification impact on the liquefaction potential mitigation. A procedure similar to that described by DAS (1992) was used. Tests were performed on top layer specimens statically compacted at their optimal water content into a mould with a diameter of 70 mm and height of 30 mm to the desired density (void ratio), and on intact specimens from top layer. The specimens were sheathed in a protective latex membrane without reinforcement.

Saturation was achieved by using the CO_2, water, and axis translation technique. The saturation was successfully completed when the Skempton parameter B was higher than 0.95. After saturation, the specimens were isotropically consolidated at 75 kPa of effective stress. The specimens were loaded with horizontal sinusoidal load at 0.5 Hz at desired cyclic stress ratio in undrained conditions (Fig. 3, right). The beginning of liquefaction was defined when the increase of pore water pressure was equal to 95% of effective vertical stress before cyclic loading.

Liquefaction potential of the top layer was evaluated also based on the results of in-situ Standard penetration tests (SPT) (SIST EN ISO 22476-3), Cone penetration tests (CPT/CPTu) (SIST EN ISO 22476-1), Flat dilatometer tests (DMT) (SIST-TS CEN ISO/TS 22476-11) and shear wave velocity measurements (v_s). SPT and CPT are generally preferred for the evaluation of liquefaction potential because of the more extensive database and experience (YOUD et al., 2001). While the preferred SPT and CPT are recommended only for non-gravel soils, v_s measurements are appropriate for all soil types (ANDRUS et al., 2004).

4. RESULTS AND DISCUSSION

4.1. Index properties – preliminary assessment of liquefaction potential

Table 2 presents the index geotechnical properties of representative top layer specimens, and Fig. 4 (grey curves) shows the grain size distribution curves of all the investigated specimens. The grain size distribution of the top layer is not uniform and may vary from sandy silt to poorly graded sand. After ISHIHARA et al. (1980), the top layer belongs to the group of potentially liquefiable to most liquefiable soils. In comparison with the investigated samples, poorly graded sands from Nova loza and Moj dvor, where liquefaction appeared in 1880, belong to the group of most liquefiable soils (VEINOVIĆ et al., 2007). The Toyoura

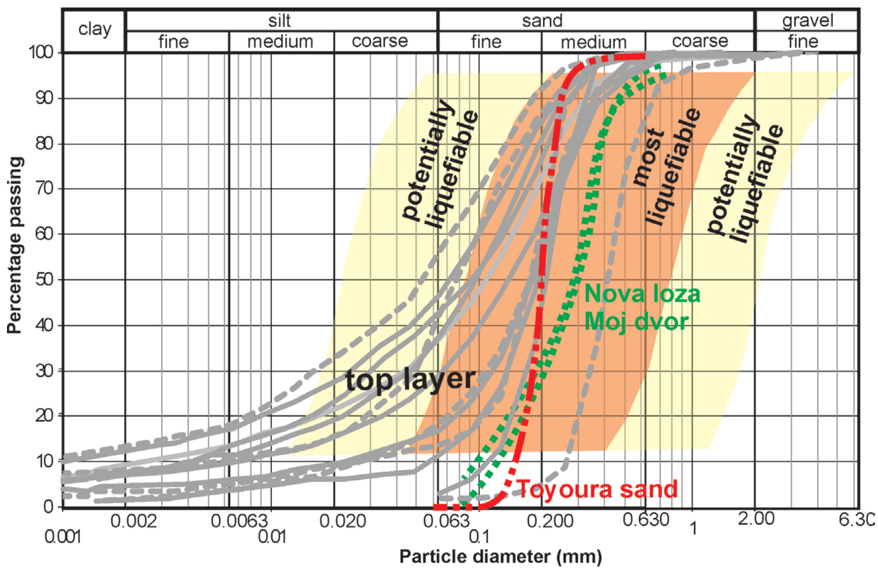

Figure 4. Grain size distribution of the top layer from the test site (grey curves). Grain size distribution curves for Toyoura sand, Nova loza and Moj dvor are reproduced from HOSONO & YOSHIMINE (2004) and VEINOVIĆ et al. (2007), while boundaries for potentially liquefiable and most liquefiable soils are reproduced from ISHIHARA et al. (1980).

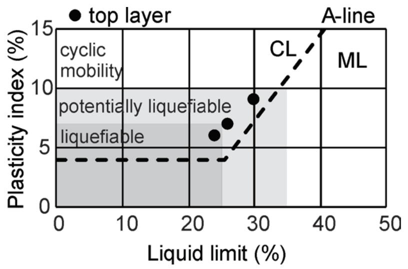

Figure 5. Assessment of the liquefaction potential based on plasticity liquefaction criteria proposed by POLITO (2001).

sand also belongs to the same group, and was in the past widely used as reference material for the study of liquefaction. Using plasticity liquefaction criteria proposed by POLITO (2001), the top layer also belongs to the group of potentially liquefiable to liquefiable soils (Fig. 5).

4.2. Laboratory cyclic simple shear tests

Figure 6 shows the results of cyclic simple shear tests as a link between the induced cyclic stress ratio (CSR) and measured number of cycles to liquefaction. The tests were carried out on intact specimens and specimens prepared at different degrees of compaction (D_{pr}), expressed as the ratio between achieved dry density and maximum SPCT dry density given in Table 2.

$CSR_{M=7.5,\sigma_v'=1}$ at the investigated area is about 0.30 (Fig. 6, horizontal solid grey line), while the corresponding equivalent number of stress cycles to liquefaction (N), determined after KRAMER (1996), for an earthquake with a magnitude of 7.5 is about 15 (Fig. 6, vertical solid grey line).

Results of the laboratory cyclic simple shear tests confirm that the intact top layer is prone to liquefaction for an earthquake with a 475 year return period. Results obtained on intact top layer specimens are comparable with those for the Toyoura sand (HOSONO & YOSHIMINE, 2004). Mechanically improved specimens compacted to at least 95% of maximum Proctor dry density (Table 2) will very probably resist the 475 year return period earthquake, although high excess pore pressure would be generated.

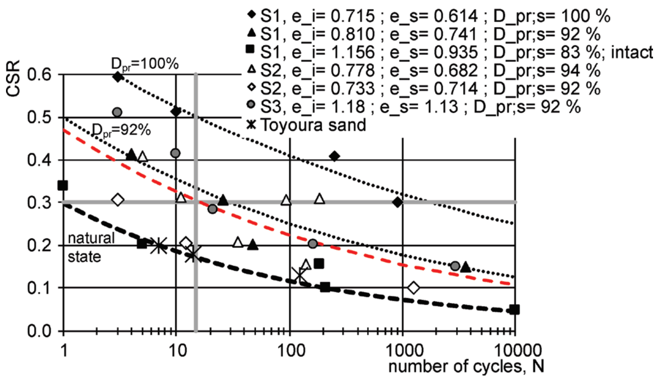

Figure 6. Results of cyclic simple shear tests. Data for the Toyoura sand were summarized after HOSONO & YOSHIMINE (2004). Key: S1, S2, S3 – sample 1, sample 2 and sample 3, e_i – initial void ratio (at preparation of the specimen), e_s – void ratio prior to cyclic loading (after consolidation), D_pr;s – the ratio between specimens' dry density prior to cyclic loading and maximum SPCT dry density, red curve – assessed boundary between conditions where liquefaction is expected (below curve) and where resistant to liquefaction is sufficient (above curve).

These findings are valid for the horizontal ground without surface loading and with the GW level at the foundation ground (e.g. 0.1 m below ground surface).

4.3. Standard penetration tests (SPT)

Figure 7 (red symbols) shows $(N_1)_{60}$ values calculated from SPT measurements in the top layer at the location of the Brežice HPP, in comparison with SPT based liquefaction case history data, including soils with different contents of fines (IDRISS & BOULANGER, 2010). Recommended deterministic SPT based triggering correlations (boundary curves) for cohesionless soils having various amounts of fines (FC), presented in Fig. 7, are summarized after IDRISS & BOULANGER (2006).

Results of SPT confirm that the top layer is prone to liquefaction for an earthquake with a 475 year return period.

4.4. Cone penetration tests (CPT/CPTu)

Figure 8 shows the q_{c1N} values calculated from CPT measurements in the top layer at the location of the Brežice HPP in comparison with CPT based liquefaction case history data, including soils with different contents of fines (BOULANGER & IDRISS, 2014). Recommended deterministic CPT based triggering correlations (boundary curves) for clean sands and for cohesionless soils with various amounts of fines (FC) presented in Fig. 8 are summarized after BOULANGER & IDRISS (2014).

Results of CPT also confirm that the top layer is prone to liquefaction for an earthquake with a 475 year return period.

4.5. Flat dilatometer tests (DMT)

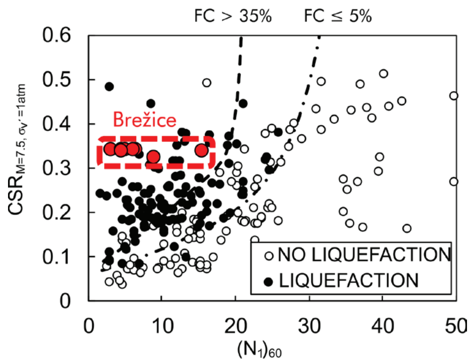

Figure 7. SPT – based soil liquefaction potential assessment with the case history data and recommended boundary curves for soils with different fines contents (FC). $(N_1)_{60}$ – corrected SPT N value for 60% energy efficiency and overburden pressure of 1 atm.

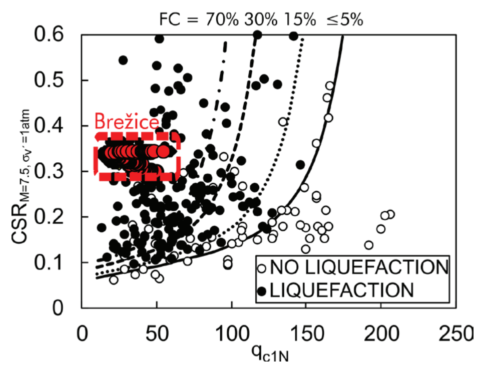

Figure 8. CPT – based soil liquefaction potential assessment with the case history data and recommended boundary curves for soils with different fines content (FC). q_{c1N} – normalized cone tip resistance.

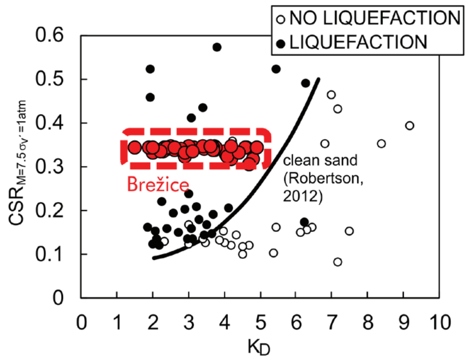

Figure 9. DMT – based soil liquefaction potential assessment with the case history data and recommended boundary curve for clean sand. K_D - horizontal stress index.

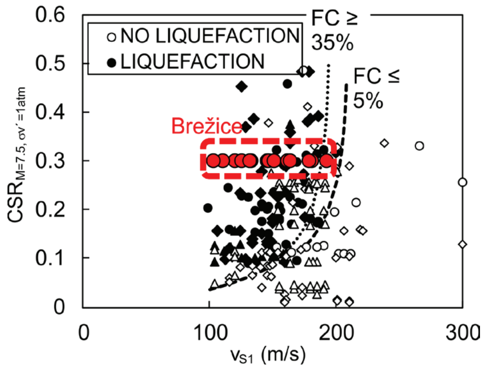

Figure 10. Liquefaction resistance curves proposed by ANDRUS & STOKOE (2000) with case history data and calculated values for the top layer at the location of the HPP Brežice. v_{s1} - stress-corrected shear-wave velocity.

Figure 9 shows the K_D measured values using DMT in the top layer at the location of the HPP Brežice in comparison with DMT-K_D based liquefaction case history data for silty sand and sandy silt, regardless of the fines content (MARCHETTI & MARCHETTI, 2016, ROLLINS & REMUND, 2016). Recommended deterministic DMT based triggering correlation (boundary curve) for clean sand presented in Fig. 9 is summarized after ROBERTSON (2012).

Results of DMT also show that the top layer is prone to liquefaction for an earthquake with a 475 year return period.

4.6. Shear wave velocity measurements (v_s)

Figure 10 shows the v_{s1} values calculated from shear wave velocity measurements in the top layer at the location of the Brežice HPP in comparison with liquefaction resistance curves proposed by ANDRUS & STOKOE (2000) and case history data (ANDRUS et al., 2004).

Shear wave velocity measurements also confirm that the top layer is prone to liquefaction for an earthquake with a 475 year return period.

5. CONCLUSIONS

The liquefaction potential of the top layer at the Krško-Brežice field was assessed using laboratory and field tests. Results were compared with case history data and recommended boundary liquefaction curves and were evaluated using liquefaction susceptibility criteria proposed by different authors.

It was observed that there are no significant differences in the evaluation of liquefaction potential based on the various test methods. The analyses indicate that there is a clear threat of liquefaction in the top layer, assuming a water table 0.1 m below the ground surface for an earthquake with a 475 year return period. Thus, for liquefaction potential mitigation, the sites require ground improvement (densification).

The efficiency of in-situ mechanical improvement for the liquefaction potential mitigation of the top layer was investigated in the scope of the design of the Brežice HPP, using three different techniques: Vibratory Roller Compaction, Rapid Impact Compaction (RIC) and Soil Mixing. The efficiency of each technique was analysed in PETKOVŠEK et al. (2017) and VUKADIN (2013).

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