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# Liquefaction Probabilities Of Semani Area, In Albania Corresponding To Different Levels Of Safety

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## ABSTRACT

In this paper the liquefaction potential of Quaternary soft non-cohesive soils at Semani site, Fier prefecture in Albania has been assessed.

The liquefaction analyses have been computed considering the hazard level corresponding to different levels of safety, such as 10% in 10 years (72-years Return Period), 10% in 50 years (475-years RP) and 2% in 50 years (2475-years RP). The accelerations corresponding to the abovementioned hazard levels are calculated using PSHA (probabilistic seismic hazard assessment), and The Cyclic Stress Ratio is calculated for each acceleration. The seismological data used in the analyses considers earthquakes with $M_S \geq 4.5$ and covers a time span from 58 up to 2009.

The calculation of cyclic resistance ratio (CRR) of the soils is based on 416 SPT (Standard Penetration Test) values, 12 CPTU tests up to 25 m depth and shear wave velocity (Vs) measurements.

The liquefaction probabilities for a given seismic event are combined with the probability of occurrence of the seismic event in order to obtain the real probability of liquefaction during the life span of the structures.

## INTRODUCTION

The study area is situated at the South West of the village Hoxhara, near the Adriatic coastline. The Quaternary's deposits composed of marshy and maritime deposits are represented by gravel, sand, silty sand, silty clay, and clays. These layers are slightly or normally consolidated. These soft deposits have a thickness of more than 100.00 m with ground water table near the ground level (0.5 m to 1.0 m). Within the first 20 m, two main porous aquifers bound to sandy soils exist, presenting specific seasonal variations. These geological and hydrogeological conditions are put into question if during strong earthquakes, like in other places with soft soils, liquefaction can take place. During the earthquake of Fier (18/03/1962), with Ms = 6.2, liquefaction phenomena were observed causing lateral spreading up to 40 cm with length reaching up to 100 m. This earthquake caused 35 victims.

A geotechnical model of study area is made based on the geotechnical investigations conducted in July - August 2006. The geotechnical investigation consists in 12 boreholes (ten borings of a depth 30 m and two borings of a depth 80.m), 416 SPT (Standard Penetration Test) N - values, 12 CPTU tests up to 25 m depth and shear wave velocity (Vs) measurements (Figure 1). These geotechnical investigations cover an area of about 216 Hectares.

For the first twenty meters this model is as follows:

Top soil and made ground represented by Silty Clay with organic matter encountered at depths 0.8 to 1.0 m from the ground level. Below this layer a loose green grey fine to medium SAND containing beds of silty sand and organic matters is encountered at the depths 2.0 m to 6.5 m. From 4.5 to 15 m soft green grey silty CLAY containing beds of silty sand, organic matters and fragments of shells is encountered. From 15 m to 18 m medium dense green grey fine to medium SAND are encountered. The rest from 18 m to 20 is composed by soft silty clays [1].

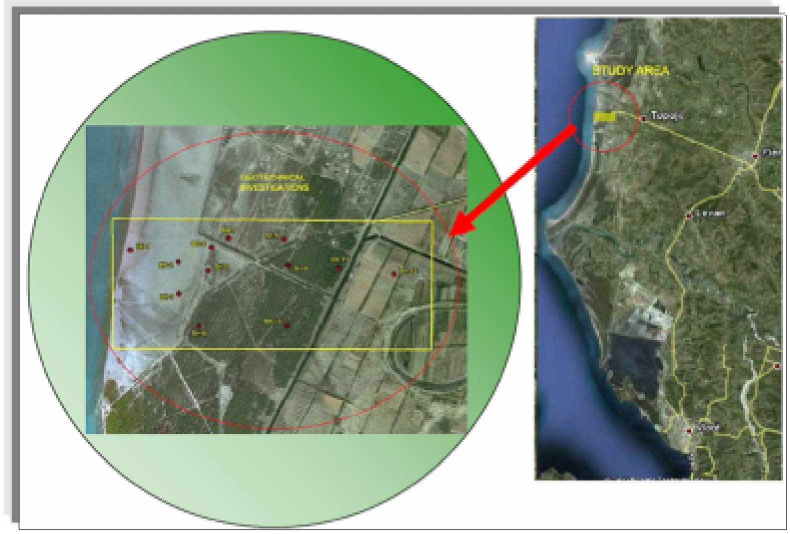


Figure 1. Geographic location of the study area

## WHY THE LIQUEFACTION ANALYSES IN THE SEMANI AREA?

The phenomenon of dynamic liquefaction occurs in sand deposits, silt and fine gravel of late Pleistocene or Holocene age, with water table located less than 15 m below the surface.

The areas at risk of liquefaction are those associated with sandy and silty soils of low plasticity and density. The cohesive soils with fine content (particles <0.005 mm in diameter) greater than 15% are generally considered not liquefiable. Some gravely soils are vulnerable to liquefaction if interposed between layers that prevent the rapid dissipation of pore pressure induced by the earthquake.

A preliminary qualitative analysis of the risk of liquefaction can be made on the basis of the following conditions [2]:

- Ground water table near the ground level,
- Holocene deposits (sand, coarse sand, fine sand, silty sand and sandy silt),
- Evidence of ancient liquefaction phenomena,
- Seismic activity in the area,
- Depth of liquefiable layers less than 20 m, however for vertical stress less than 200 kPa.

All the above mentioned conditions are fulfilled for the study area and consequently the soils can be considered susceptible to liquefaction.

## LIQUEFACTION ASSESSMENT AT SEMANI AREA

- **Cyclic Stress Ratio, CSR**

The cyclic stress ratio, CSR, is a measure of the seismic induced stress, expressed as equivalent uniform cyclic stress, divided by the initial effective overburden pressure. The average uniform cyclic stress ratio within a liquefiable stratum at representative depth of the dynamic loading imposed by the earthquake is given by Seed and Idriss (1971):

$$CSR = \frac{\tau_{av}}{\sigma_{vo}} = 0.65 \cdot \left(\frac{a_{\max}}{g}\right) \cdot \left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right) \cdot r_d$$

where, CSR is the Cyclic shear Stress Ratio; $\sigma_{vo}$ - total overburden pressure; $\sigma'_{vo}$ - effective overburden pressure; $a_{max}$ - maximum surface acceleration in units of g; g - gravity acceleration and $r_d$ is a stress reduction factor which is dependent on depth.

Three values of $a_{max}$, corresponding to three hazard levels are used in CSR calculation. For the first hazard level (earthquakes with 50% probability of being exceeded during the life-span of a structure) the evaluated maximum acceleration is $a_{max}$=0.25 g; for the second hazard level (earthquakes with 10% probability of being exceeded during the life-span of a structure) the evaluated maximum acceleration is $a_{max}$=0.388 g while for the third hazard level (earthquakes with 2% probability of being exceeded during the life-span of a structure) the evaluated maximum acceleration is $a_{max}$=0.487 g [3].

The total ($\sigma_v$) and effective ($\sigma_{v'}$) vertical stresses are estimated from the geotechnical data available for the site.

The relations proposed by Liao and Whitman (1986) use a function of $r_d$ that is dependent only on soil depth as follow:

$r_d = 1.0 - 0.00765z$ if z<9.15 m

$r_d = 1.174 - 0.0267z$ if z=9.15 to 23 m

The CSR is corrected by an additional safety factor ($f_{sl}$), which typical value is 1.2, for taking into account the influence of the pore pressure changes and strain developed during the ground motion (*CSR fs*). A soil is predicted to liquefy if $F_s \leq 1.2$ (Sonmez, 2003).

$$CSRfs = CSR \cdot f_{sl}$$

- **Cyclic Resistance Ratio, CRR**

The cyclic resistance ratio is calculated for an earthquake of $M_s$=6.1 and the calculation are based on CPT, SPT and VS measurements.

- ***Soil profile from CPT data***

From the CPT-data the soil type index, *Ic*, defined by Robertson and Wride (1998), permits to obtain a detailed lithological depth profile using the following equation:

$$I_c = \left[ (3.47 - \log Q)^2 + (1.22 + \log F)^2 \right]^{0.5}$$

Where:

$Q$ = the normalized cone penetration resistance, dimensionless

$$Q = \left[ (q_c - \sigma_{v0}) / P_a \right] * \left[ (P_a / \sigma'_{v0})^n \right].$$

With:

$\sigma_{v0}$ and $\sigma'_{v0}$ are the initial total and effective overburden stresses, respectively; $P_a$ is a reference pressure in the same units as $\sigma'_{v0}$, $q_c$ and $\sigma_{v0}$.

The stress exponent $n$, in the formula for calculating the resistance of the soil $Q$ varies in relation to soil type. For clean sands the value is 0.5, silt and silty sands an appropriate value is between 0.5 and 1.0 ($n$ = 1 characteristic of the clay). The iteration procedure proposed by Robertson (1990) is used to evaluate $n$ and the $I_c$ index used to define the soil type.

$F$ = normalized friction ratio defined as $F = \left[ f_s / (q_c - \sigma_{v0}) \right] * 100\%$

$f_s$ is the CPT sleeve friction stress and $q_c$ is the measured soil resistance.

The soil type index $I_c$ helps among others to restrict the probability of liquefaction to soils with $I_c < 2.8$ (Yuang et al., 2003)

- ***Cyclic Resistance Ratio, CRR7.5 based on CPT data***

The cyclic resistance ratio of clean sands, for a magnitude of 7.5 (*CRR7.5*), after Robertson and Wride (1998) has the following equations:

$$CRR_{7.5} = 0.833\left[\left(q_{c1N}\right)_{cs}/1000\right] + 0.05 \qquad for\left(q_{c1N}\right)_{cs} < 50$$

$$CRR_{7.5} = 93\left[\left(q_{c1N}\right)_{cs}/1000\right]^3 + 0.08 \qquad for 50 \le \left(q_{c1N}\right)_{cs} < 160$$

With $(q_{c1N})_{CS}$ the normalized cone penetration resistance corrected for the fine content influence as follow:

$$\left(q_{c1N}\right)_{cs} = K_c * q_{c1N}$$

Where $K_c$ is the correction factor for particle size characteristics, defined by the following equation (Robertson and Wride, 1998):

$$K_c = 1.0 \qquad I_c \le 1.64$$

$$K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88 \qquad I_c > 1.64$$

and $(q_{c1N})$ is the normalized (stress-adjusted) cone penetration resistance defined as follow: .

$$\left(q_{c1N}\right)_{cs} = C_q * (q_c / P_a)$$

$C_q = (P_a / \sigma'_{v0})^n$, normalized tip resistance factor

- ***Cyclic Resistance Ratio (CRR) based on SPT data***

The N-value of SPT, measures the penetration resistance of the granular soil, which is directly related to its liquefaction resistance (*CRR*).

The N-value of SPT is normalized to a hammer energy ratio of 60% $(N)_{60}$, while the $(N1)_{60}$, used as a dependent variable in the probability relations, represents the normalized penetration resistance of the soil under an effective overburden pressure of 100 kPa.

$$(N_1)_{60} = (N_{SPT} \cdot C_E \cdot C_B \cdot C_R \cdot C_S) \cdot C_N = (N)_{60} \cdot C_N$$

where, $N_{SPT}$ is N-value of SPT; $C_E$ - hammer energy efficiency correction; $C_B$ -borehole diameter correction; $C_R$ - "short" rod length correction; $C_S$ - non-standardized sampler configuration correction and $C_N$ is the effective overburden pressure correction.

According to Liao and Whitman (1986) the $C_N$ is estimated as below:

$$C_N = \left(\frac{Pa}{\sigma'_{vo}}\right)^{0.5}$$

Where, $\sigma'_{vo}$ is the actual effective overburden stress at the depth of the SPT.

For the purposes of assessing susceptibility to liquefaction, it is important to consider not only the severity of the ground motion, as quantified by the ground motion acceleration, $a_{max}$, but also the duration of shaking. It is known the earthquake magnitude is an appropriate dependent variable for any functional parameterization of duration. For this purpose, Rauch (1998) presents a magnitude (7.5) normalized form of the cyclic resistance ratio ($CRR_{7.5}$).

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

Where, $\left(N_1\right)_{60cs} = \alpha + \beta\cdot\left(N_1\right)_{60}$ as proposed by Youd and Idriss (1997) for the influence of fine content (FC) in the normalized cyclic resistance ratio, with $\alpha$ and $\beta$ given as follow: for FC < 5 % $\alpha = 0$ and $\beta = 1$; for FC > 35% $\alpha = 5$ and $\beta = 1.2$; for FC between 5% and 35% the parameters are given by the following equations:

$$\alpha = \exp\left[1.76 - \left(190/FC^2\right)\right]$$

$$\beta = 0.99 + FC^{1.5}/1000$$

- ***Cyclic Resistance Ratio (CRR) based on share wave velocity ($V_S$)***

The resistance of the soil to liquefaction, expressed as a cyclic resistance ratio (*CRR*) can be estimated by the equation proposed by Andrus and Stokoe (1997) as follow:

$$CRR = \left[ a\left(\frac{V_{s1}}{100}\right)^2 + b\left(\frac{1}{V_{s1}^* - V_{s1}} - \frac{1}{V_{s1}^*}\right)\right]$$

Where $V^*$ = limiting upper value of $V_{S1}$ for cyclic liquefaction occurrence; $a$ and $b$ = curve fitting parameters respectively 0.022 and 2.8.

Values of $V^*_{S1}$ are assumed to range linearly from 200 to 215 m/s. The relationship between $V^*_{S1}$ and fines content can be expressed by:

$V^*_{S1}$ = 215 m/s for sands with FC up to 5%; $V^*_{S1} = 215 - 0.5(FC-5)$ m/s for sands with FC between 5% and 35 % and $V^*_{S1}$ = 200 for sand with FC greater or equal than 35 %.

$V_{s1}$ is overburden stress-corrected shear-wave velocity defined by Sykora (1987); Robertson et al. (1992) as,

$$V_{S1} = V_s C_v = V_s \left(\frac{p_a}{\sigma_v^{'}}\right)^{0.25}$$

$C_V$ = factor to correct measured shear-wave velocity for overburden pressure; for shallow depths the maximum $C_V$ value of 1.4 is generally applied to $V_S$ data.

- **Magnitude Scaling Factors**

The *CRR* value calculated for an earthquake of magnitude of 7.5. The cyclic resistance ratio for the given magnitude ($CRR_M$) is calculated by introducing the magnitude scaling factor (*MSF*) as follow:

$$CRR_M = CRR_{7.5} \cdot MSF$$

This factor is traditionally applied to *CRR*, and equals 1.0 for earthquakes with a magnitude of 7.5. For magnitudes other than 7.5, magnitude scaling factors are developed by various investigators.

For the *CRR* calculation for the given magnitude the following relations are used:

- For *CRR* calculated using CPT and SPT data, for earthquakes with magnitude lower than 7.5 the relation proposed by Youd et al (2001) is used , $MSF = 10^{2.84} / M^{3.24}$.
- For *CRR* calculated using shear wave velocities ($V_S$) the relation proposed by Andrus & Stokoe (1997) is used: $MSF = \left[\frac{M}{7.5}\right]^{-2.56}$

Further correction was made to $CRR_M$ for the shear resistance and the effective lithostatic stress as proposed by Idriss (1983). The equivalent Cyclic Resistance Ratio ($CRR_{eq}$) is obtained as follows: $CRR_{eq} = CRR_M \cdot K\sigma \cdot K\alpha$ , with $K\alpha = 1$ for horizontal areas.

NCCER (Youd, 1997) suggested:

for $\sigma_{v0}^{'} / P_a \leq 1$ $\quad K_\sigma = 1$; and

for $\sigma_{v0}^{'} / P_a > 1$ $\quad K_\sigma = (\sigma_{v0}^{'} / P_a)^{-0.25}$

- **Safety Factor**

The Safety Factor of Liquefaction, FSL, is expressed as the ratio of Cyclic Stress Ratio (demand) and the Cyclic Resistance Ratio (capacity):

$$FS_L = \frac{CRR_{eq}}{CSR_{fs}}$$

## PROBABILITY CALCULATION

Probability of liquefaction assessment involves two stages: 1) calculation of liquefaction for a given seismic event, and 2) combination with the probability that this event occurs.

- **Liquefaction probability**

Based on conventional probability theory, the probability of liquefaction, for the given seismic event, is calculated from the $FS_L$ using different correlations proposed by several authors.

In this study the relation proposed by Chen and Juang (2000) is used for the liquefaction probability SPT and *Vs* based $F_{SL}$ calculation, while for CPT based $F_{SL}$ calculation it is used the correlation proposed by Juang et al (2006).

$$P\left[Liquefaction\,|_{PGA=a}\right] = \frac{1}{1+(FS_L/0.77)^{3.25}} \qquad \text{Chen and Juang }(2000)$$

$$P\left[Liquefaction\,|_{PGA=a}\right] = \frac{1}{1+(FS_L/0.74)^{5.45}} \qquad \text{Juang }(2006)$$

The likelihood of liquefaction can be interpreted using the calculated $P_L$ values in Table 1. It can be seen in Table 1, that liquefaction will occur only if the probability of liquefaction is greater than 35%. The calculated probability permits to observe if a layer is susceptible to liquefy during a specific earthquake.

Table 1. The classification of probability of liquefaction

| Probability | Likelihood of liquefaction |
|---|---|
| 0.85 ≤ PL <1 | Almost certain that it will liquefy |
| 0.65 ≤ PL <0.85 | Very likely |
| 0.35 ≤ PL <0.65 | Liquefaction/non-liquefaction is equally likely |
| 0.15 ≤ PL <0.35 | Unlikely |
| 0.00 ≤ PL <0.15 | Almost certain that it will not liquefy |

- **Conditional (Real) Probability**

Given an evaluation of the probability of liquefaction occurring from a seismic event, $P\left[Liquefaction\,|_{PGA=a}\right]$ and given the probability that the seismic event occurs, $P\left[E\,|_{PGA=a}\right]$, we may estimate the joint probability as follow:

$$P[L] = P\left[Liquefaction\,|_{PGA=a}\right] * P\left[E\,|_{PGA=a}\right]$$

## RESULTS

The liquefaction probabilities, $\mathrm{P}\left[\mathrm{Liquefaction}\,|_{\mathrm{PGA=a}}\right]$ of the sandy soil layers for each borehole are calculated based on CPTU and SPT results, whereas a simplified model was used for the calculation of $V_S$ - based probabilities.

The results show that layers susceptible to liquefaction are liquefiable for the first level of seismic hazard, to which corresponds an acceleration of PGA = 0.25g. The sandy layers are encountered in the depth intervals from 1.2 to 6-7 m, reaching a maximum down to 10 m (Figure 2). The second interval of the liquefiable layers is found from 15 to 20 m. For this acceleration, the calculated probabilities based on CPTU, SPT and Vs data have higher values than 35%. Based on liquefaction probability classification according to Chen and Juang (2000), for these layers liquefaction/non-liquefaction is equally likely. In-between these layers however, there were encountered levels for which liquefaction is very likely or almost certain.

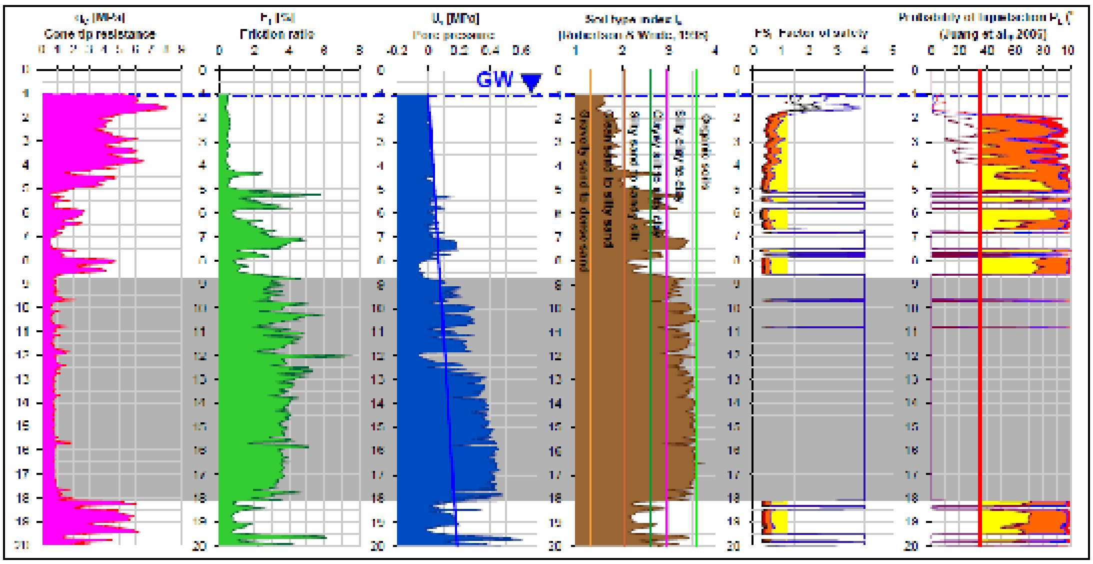


Figure 2 Example of liquefaction probability calculation based on CPT data

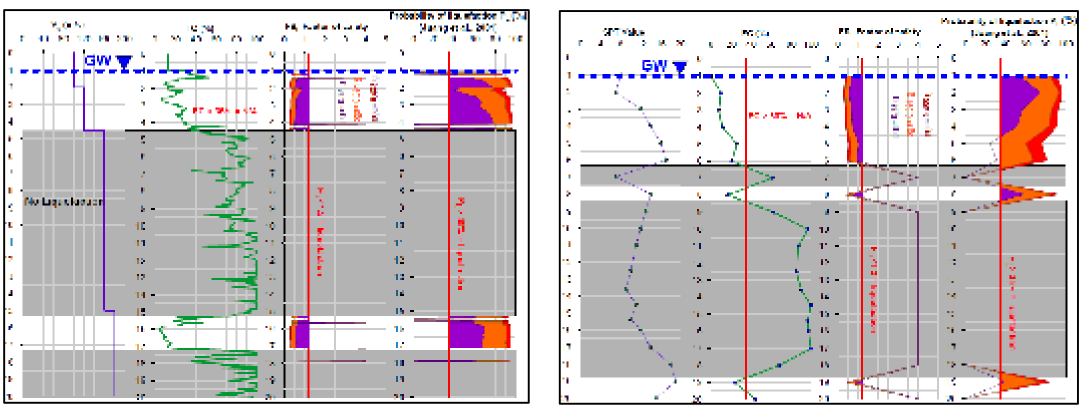


Figure 3. Liquefaction probability calculation based on *Vs* model (left) and SPT data (right)

Considering the first level of hazard with $a_{max}$=0.25 g (50% probability of exceedance during the life span of the structures) the real probability of liquefaction P[L] of the liquefiable soils reaches values of varies from 15 % to 40-45 %. For the second and third level of hazard even though the liquefaction probabilities increase, the real probabilities decrease due to the smallest possibility of the occurrence of such seismic events.

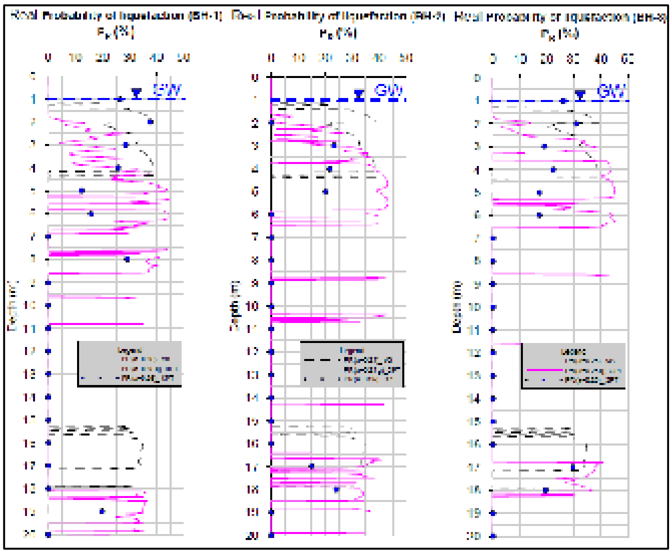

Figure 4. Real probability of liquefaction.

## CONCLUSIONS

For the first time a liquefaction risk analysis for the Semani area was performed. The CPT-field data executed in 2006 were of high quality. We cannot say the same thing for SPT tests and measurements Vs.
The investigations reached optimal depths of 30 m, allowing the accurate evaluation of the liquefaction probability and the real probability at Semani area.

The sandy layers encountered in the first 20 m are susceptible to liquefaction. The real probabilities calculated for the first hazard level reaches values of 40 %, so during the life span of structure liquefaction phenomenon may be observed.
The $V_S$ based probabilities are similar to those calculated via CPT whereas the probabilities calculated via SPT are slightly lower for depths higher than 10 m.

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