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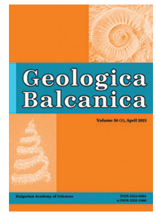

Liquefaction potential assessment of saturated loess

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Abstract. Usually, soils with mainly fine grain-sized content, as loess, are considered to have low liquefaction potential. Regardless of this, many researchers have analyzed and presented much field evidence that silty soil (in particular loess) liquefaction occurred under certain conditions. In Bulgaria, the first loess river terrace (T_1) within the Danube River lowland areas is covered by low plasticity silty loess with a thickness of 10–12 m. The groundwater level is often located between 5 m and 8 m in depth so that substantial part of loess deposits are saturated and immersed. Meanwhile, that region of North Bulgaria is under the influence of the Vrancea seismic zone in Romania, which is able to generate strong earthquakes with magnitudes M≥7.0. The present paper aims to assess the liquefaction potential of loess in a ground profile representative of the T_1 loess river terraces by the so-called simplified procedure based on SPT, which is incorporated in the software code *NovoLiq*. The safety factor against liquefaction FS_L is estimated at the respective depths in one-dimensional model of the ground profile for free-field conditions at varying peak ground accelerations a_{max}. The critical a_{max}, at which liquefaction of loess is possible according to the assumptions of the applied simplified procedure and the requirements of the National Annex of Bulgaria to Eurocode 8, has been established.

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Keywords: loess, T_1 river terrace, lowland areas along the Danube River (Bulgaria), SPT, liquefaction potential.

INTRODUCTION

Usually, soils with mainly fine grain-sized content, as loess, are considered to have low liquefaction potential. Regardless of this, many researchers have analyzed and provided numerous field examples, mostly from China, that loess liquefaction occurred under certain conditions (*e.g.*, Wang *et al.*, 2004, 2014; Pei *et al.*, 2017). Observations following earthquakes in China indicated that many cohesive soils had liquefied (Wang, 1979, 1984). These cohesive soils had clay fraction less than 20%, liquid limit (LL) between 21% and 35%, plasticity index (PI) between 4% and 14%, and water content (w) greater than 90% of their liquid limit (Wang, 1979). Liquefaction of soils with up to 70% fines and 10% clay fraction during the Mino-Owar, Tohankai, and Fukui earthquakes was reported by Kishida (1969). Wang *et al.* (2007) investigated the liquefaction susceptibility of saturated loess (with PI from 7.2% to 9%) and fine sand from an airport site near Lanzhou, China. Their study showed that loess was more susceptible to liquefaction than fine sand.

Perlea *et al.* (1999) adapted the Chinese criteria to ASTM definitions of soil properties (Fig. 1). Soils that fall below the line defined by w = 0.87×LL and LL = 33.5% in Fig. 1 are considered susceptible to liquefaction.

Seed *et al.* (1983) suggested that some soils with fines (<0.075 mm) may be susceptible to lique-

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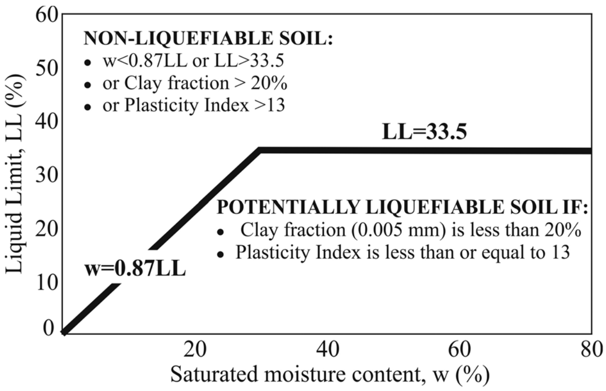

Fig 1. Chinese criteria for susceptibility to liquefaction adapted to ASTM definitions of soil properties (after Perlea *et al*., 1999).

faction if they meet the following characteristics (based on the Chinese criteria): percentage of particles finer than 0.005 mm <15 %; LL<35%; and w>0.90×LL. Based on the case histories and theory, Andrews and Martin (2000) reinforced and refined the criteria outlined by Seed *et al*. (1983) for silty soils' liquefaction susceptibility as shown in Table 1.

Karastanev (1998) reported results of cyclic undrained triaxial tests of low plasticity silt soil (typical loess from North Bulgaria). The tests were carried out with undisturbed samples having three saturation degrees S_r of 0.30, 0.54, and ≥0.96. The obtained results (Fig. 2) indicated considerable loss of dynamic resistance of saturated loess at seismic impact. Berov *et al*. (2017) classified the loess deposits in the Danube plain (North Bulgaria) as prone to liquefaction.

Idriss and Boulanger (2008) investigated the liquefaction of soils with fines and showed that fine-grained soils with more than 50% grains passing 0.075 mm can be reasonably grouped into soils that exhibit either sand-like stress-strain behavior or clay-like stress-strain behavior during monotonic and cyclic undrained shear loading.

Prakash and Puri (2010) reviewed literature with recommendations on liquefaction of soils with fines. They concluded that silts and silt-clay mixtures can be prone to liquefaction under certain conditions, and meanwhile no definite guidelines are available to ascertain liquefaction susceptibility of fine soils based on a simple test, as for sands. Thus, more work is needed, and probably in a few decades we will have a good understanding of the liquefaction behavior of fine grained soils.

In Bulgaria, loess forms an almost continuous cover in North Bulgaria south of the Danube River. It was formed during the Pleistocene and has eolian genesis (Evlogiev, 2019). In general, the main geotechnical properties of the Bulgarian loess are similar to loess deposits in other parts of the world, *i.e.*, high content of silt particles (0.075–0.002 mm), high porosity due to the loose soil structure, high water sensitivity, and severe ground collapse induced by wetting with/without construction loading.

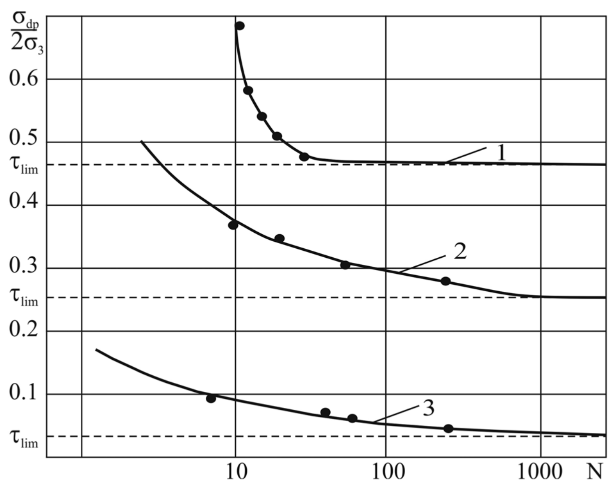

Fig. 2. Cyclic stress ratio $\sigma_{dp}/2\sigma_3$ versus number of loading cycles N causing initial failure (liquefaction) for low-plasticity silty loess with different degrees of saturation: *1* – S_r = 0.30; *2* – S_r = 0.54; *3* – S_r ≥ 0.96 (after Karastanev, 1998).

Table 1
Modified criteria of Seed et al. (1983) for susceptibility to liquefaction of silty soils (Andrews and Martin, 2000)

	LL* < 32%	LL > 32%
Clay content** <10%	susceptible	further studies required
Clay content ≥10%	further studies required	not susceptible

* LL determined by Casagrande-type apparatus
** Clay particles (<0.002 mm)

The so-called first loess river terrace (T_1) within the Danube lowland areas is covered by low plasticity silty loess with a thickness of 10–12 m (Petrova, 2009; Evlogiev, 2019). The groundwater level is often located between 5–8 m in depth so that a substantial part of the loess deposits are saturated and immersed. Meanwhile, that region of North Bulgaria is under the influence of the Vrancea seismic zone in Romania, which has been able to generate strong earthquakes with magnitudes M≥7.0.

Taking into account the above considerations, the present paper aims to assess the liquefaction potential of loess from the T_1 loess river terraces within the Danube lowlands in North Bulgaria, based on the widely used throughout the world simplified procedure. This simplified procedure was originally developed by Seed and Idriss (1971), Seed *et al*. (1983), using blow counts from the Standard Penetration Test (SPT) correlated with a parameter representing the seismic resistance of the soil (*i.e.*, the Cyclic Resistance Ratio). The procedure was slightly modified by Youd and Idriss, (1997) and Youd *et al*., (2001), and further updated by Cetin *et al*. (2004) and Idriss and Boulanger (2008, 2010). The simplified procedure is essentially incorporated in EN 1998-5:2004 (Fardis *et al.*, 2005) for identification of potentially liquefiable soils and assessment of liquefaction hazard.

METHODOLOGY

The assessment of the liquefaction hazard in the simplified procedure is undertaken by comparing the specific action effect at a given depth, designated as Cyclic Stress Ratio (CSR), induced by particular design earthquake with the soil resistance against liquefaction at that depth, designated as Cyclic Resistance Ratio (CRR). Hence, the ratio

$$FS_L = \frac{CRR}{CSR}$$

is denoted as safety factor against liquefaction FS_L at the respective depths in a one-dimensional model of the soil profile. At depths where FS_L<1, the hazard of liquefaction is likely, and unlikely where FS_L>1 for the particular seismic action.

The Cyclic Stress Ratio (CSR) shows the effect of the design earthquake (with a specific magnitude) at the respective ground depth. It is expressed as follows:

$$\mathrm{CSR} = 0.65\frac{\tau_{\max}}{\sigma'_{v_o}},$$

where: τ_{max} is maximum earthquake-induced shear stress [kPa]; and σ'_{v0} is effective vertical overburden stress [kPa] at the respective depth. The reference stress level of 0.65 was selected by Seed and Idriss (1967) and has been in use since then (Boulanger and Idriss, 2014). The value of τ_{max} can be estimated by different dynamic response tests, which are usually complicated and time-consuming. As alternative, Seed and Idriss (1971) formulated the following equation for CSR estimation:

$$\mathrm{CSR} = 0.65\frac{\sigma_{v_0}}{\sigma'_{v_0}}\frac{a_{\max}}{g}r_d\ ,$$

where: σ_{v0} is total vertical overburden stresses at the respective depth [kPa]; a_{max} is peak horizontal acceleration at the ground surface generated by the respective earthquake [m/s^2]; g is gravitational acceleration [m/s^2]; and r_d is shear stress reduction coefficient considering the dynamic response of the soil at the respective depth z. Youd *et al*. (2001) recommended applicable correlations $r_d = f(z)$ to estimate the average values of r_d.

The Cyclic Resistance Ratio (CRR) expresses the capacity of soil to resist liquefaction. To avoid the difficulties connected with sampling and laboratory cyclic testing, CRR is usually correlated to an *in-situ* test parameter such as SPT blow counts. Since SPT resistance is influenced by a number of test procedure details (rod lengths, hammer energy, sampler details, borehole size) and by effective overburden stress, the correlation to CRR is based on corrected or normalized SPT blow counts $N_{1(60)}$. In case of cohesionless soils with more fines content, additional correction is applied to $N_{1(60)}$. The detailed evaluation of $N_{1(60)}$ considering the respective correction factors were presented by Youd *et al*. (2001), Idriss and Boulanger (2010), and others. Details for the estimation of CRR can be found in Youd and Idriss (1997). The correlation of CRR with $N_{1(60)}$ is developed for a reference magnitude M = 7.5 and σ'_{v0} = 100 kPa. For any other value of M and σ'_{v0}, the CRR is determined by the following equation:

$$\mathrm{CRR} = \mathrm{CRR}_{M=7.5} \times \mathrm{MSF} \times k_\sigma \times k_\alpha\ ,$$

where: $\mathrm{CRR}_{M\ =\ 7.5}$ is Cyclic Resistance Ratio for M = 7.5 and the corresponding $N_{1(60)}$ in the respective depth; MSF is correction factor for magnitudes different from M = 7.5; k_σ is overburden stress correction factor (for stress >100 kPa); and k_α is ground slope correction factor.

Substantially, the described methodology is incorporated in the software code *NovoLiq* (Version

4.0.2021.212), which is designed for soil liquefaction analysis of a multilayer profile during seismic impact. This software product is applied in the current study for assessment of the liquefaction potential of loess from the Danube lowland areas in North Bulgaria.

According to EN 1998-5:2004, a soil should be considered susceptible to liquefaction under level-ground conditions whenever the earthquake-induced shear stress exceeds a certain fraction of the critical stress known to have caused liquefaction in previous earthquakes. The recommended value is $\lambda = 0.8$, which implies that the minimum value of the safety factor against liquefaction is $FS_L = 1.25$. The value ascribed to λ in the National Annex of Bulgaria to Eurocode 8 (EN 1998-1/NA. 2012) is $\lambda = 0.75$, which correspondents to $FS_L = 1.33$. This way, the uncertainties of the field procedure, in which no partial factors are applied to the *in-situ* resistance parameter $N_{1(60)}$, are taken into account (Fardis *et al.*, 2005).

ANALYZED GROUND PROFILE AND SOIL CHARACTERISTICS

In the current study, a ground profile representative of the first loess river terrace (T_1) within the Danube lowland areas was analyzed. Details of this ground profile are shown in Fig. 3. From the surface down to 10 m depth, the soil profile comprises typical loess, which classifies, according to ASTM D 2487 (USCS) USCS, as *CL – lean clay*. The fines content (<0.075 mm) is 96–98%. In general, it consists of only silt particles, with no clay (<0.002 mm) particles. In the interval of 10 m to 12 m, more clayey loess is situated, which is also classified as *CL – lean clay*. Its fine content is similar (about 95%), but it usually comprises up to 10% clay (<0.002 mm) particles. Practically, the loess is typical low plasticity silt soil with LL in the range of 30–33% and PI of 10–12%.

Below the loess complex, alluvial sediments lie, with a thickness of about 7 m, from 12 m to 19 m depth. They consist of gravelly sand with silt, and below them coarse gravel is situated. The substratum of Quaternary deposits is the Pliocene firm to stiff clay starting from a depth of 19 m.

The SPT values correspond distinctly to the ground profile (Fig. 3). In the loess, N_{SPT} values (*in-situ* values without corrections) are between 5 and 8, while in the clayey loess they are in the range of 13–15. In the alluvial deposits situated between 12 m and 19 m depth, the N_{SPT} values increase to 22–30 in the gravelly sand with silt, and to 35–54

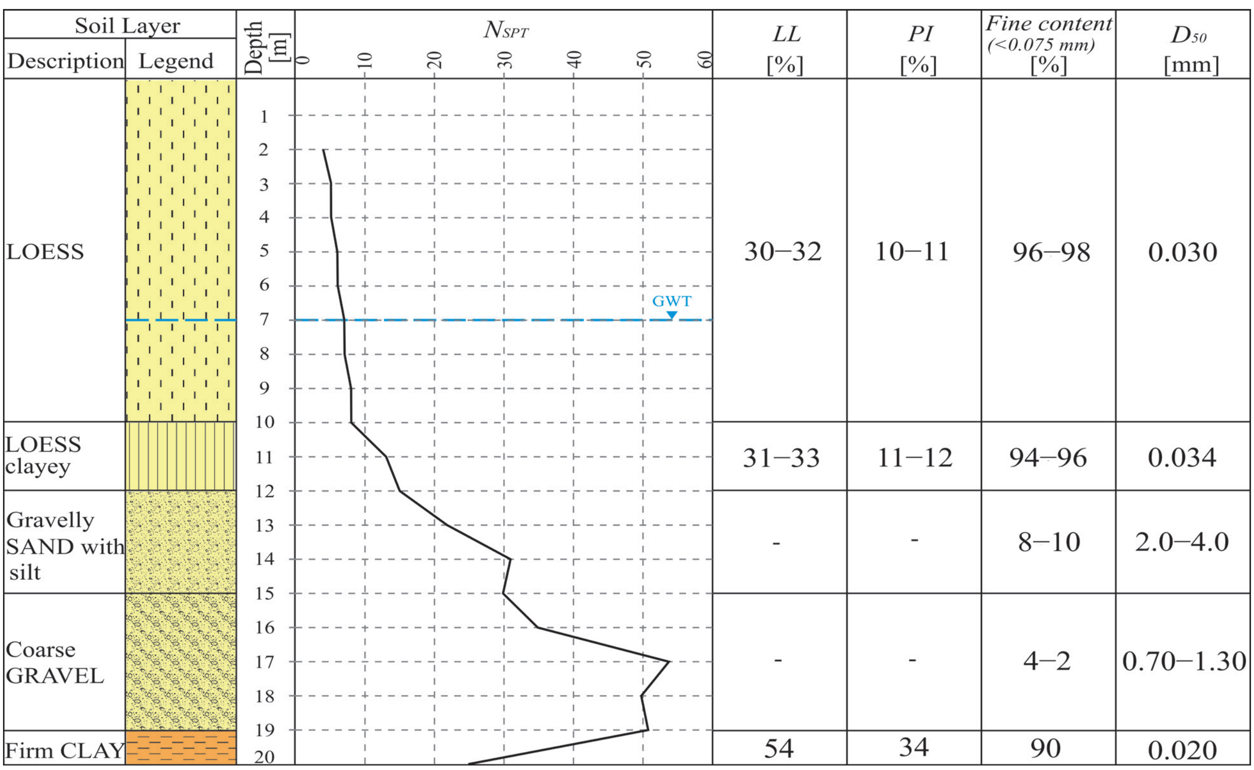

Fig. 3. Analyzed ground profile and soil characteristics.

in the coarse gravel. In the Pliocene clay, the N_{SPT} value is 25. It must be noted that the N_{SPT} values were obtained by standard penetration tests following the ASTM 1586 requirements. The tests were performed by automatic hammer drop systems with a split-barrel sampler without liner in boreholes with 200 mm diameter and metal casing.

The ground water table (GWT) normally varies between 7 m and 8 m below the ground surface, depending on the water level of the Danube River. It has to be pointed out that the loess is usually saturated several meters above GWL due to the strong capillary forces in it.

RESULTS AND DISCUSSION

The corrected SPT blow counts $N_{1(60)}$ versus depth and the respective D_r are shown in Fig. 4*a*, *b*. It can be seen that, down to 10 m depth, the relative density D_r of loess is below 60%, which indicates susceptibility to liquefaction.

The stress reduction coefficient r_d, depending on the depth *z*, is presented in Fig. 4*c*, and the total σ_{v0} and effective σ'_{v0} vertical overburden stresses acting at the respective depth are plotted in Fig. 4*d*.

$CRR_{M=7.5}$ in the respective depth was computed by the calculation method of Youd and Idriss (1997) and Youd *et al*. (2001), incorporated in *NovoLiq*. CRR is estimated for an earthquake with a magnitude of $M = 7.0$, which is assumed as an equivalent magnitude for the region of Northwest Bulgaria under the impact of the Vrancea seismic zone situated at a distance of 240 km. The MSF value is estimated to 1.193.

CSR is determined for free field conditions at varying peak ground accelerations (PGA) a_{max} in order to assess the alteration of the safety factor against liquefaction. The CSR and CRR results versus depth obtained at selected accelerations are shown in Fig. 5.

The safety factor against liquefaction FS_L is estimated by the same calculation method at the respective depths in a one-dimensional model of the analyzed ground profile. The results obtained are shown in Fig. 5*a* at $a_{max} = 0.20g$, in Fig. 5*b* at $a_{max} = 0.22g$, and in Fig. 5*c* at $a_{max} = 0.26g$. It must be noted that *NovoLiq* uses an upper limit of 3.0 when calculating FS_L by the method described by Youd and Idriss (1997) and Youd *et al*. (2001). In this case, it concerns the coarse gravel layer situated at depths below 14.0 m.

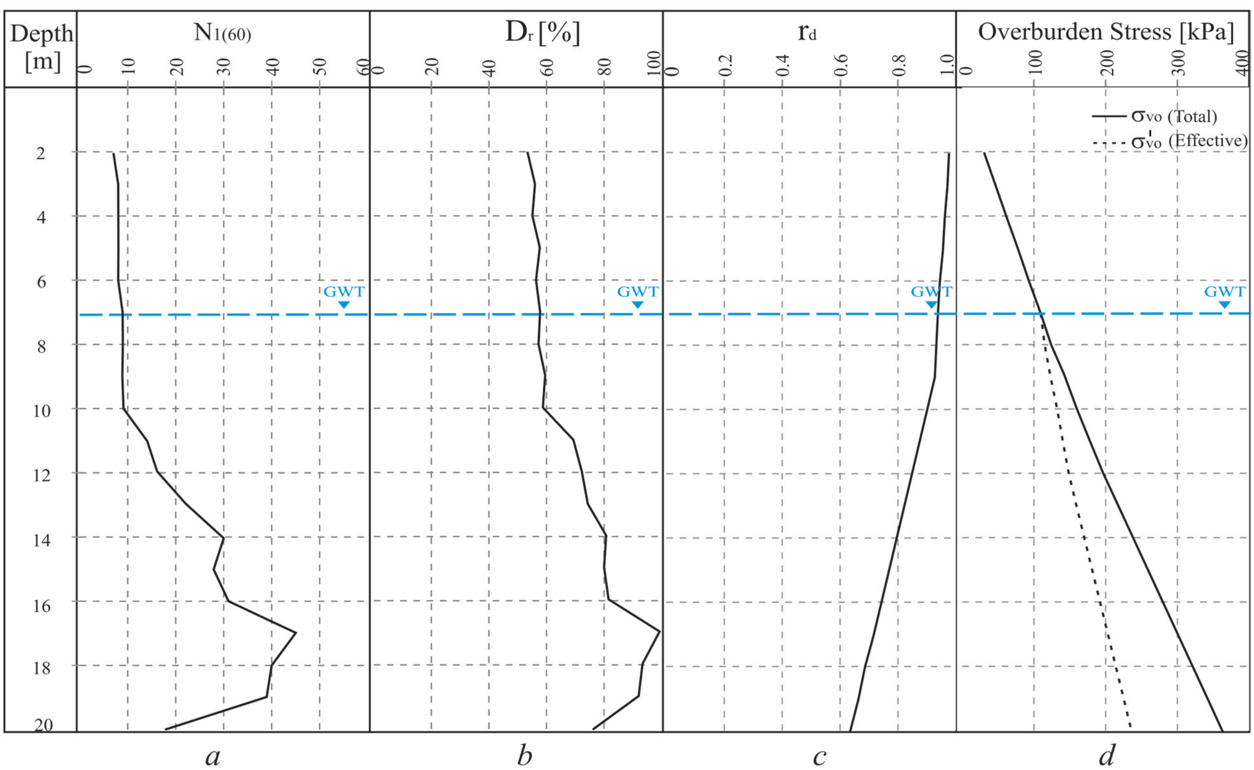

Fig. 4. Corrected (normalized) SPT blow counts $N_{1(60)}$ (*a*), relative density D_r (*b*), shear stress reduction coefficient r_d (*c*), and total and effective vertical overburden stresses (*d*) vs depth.

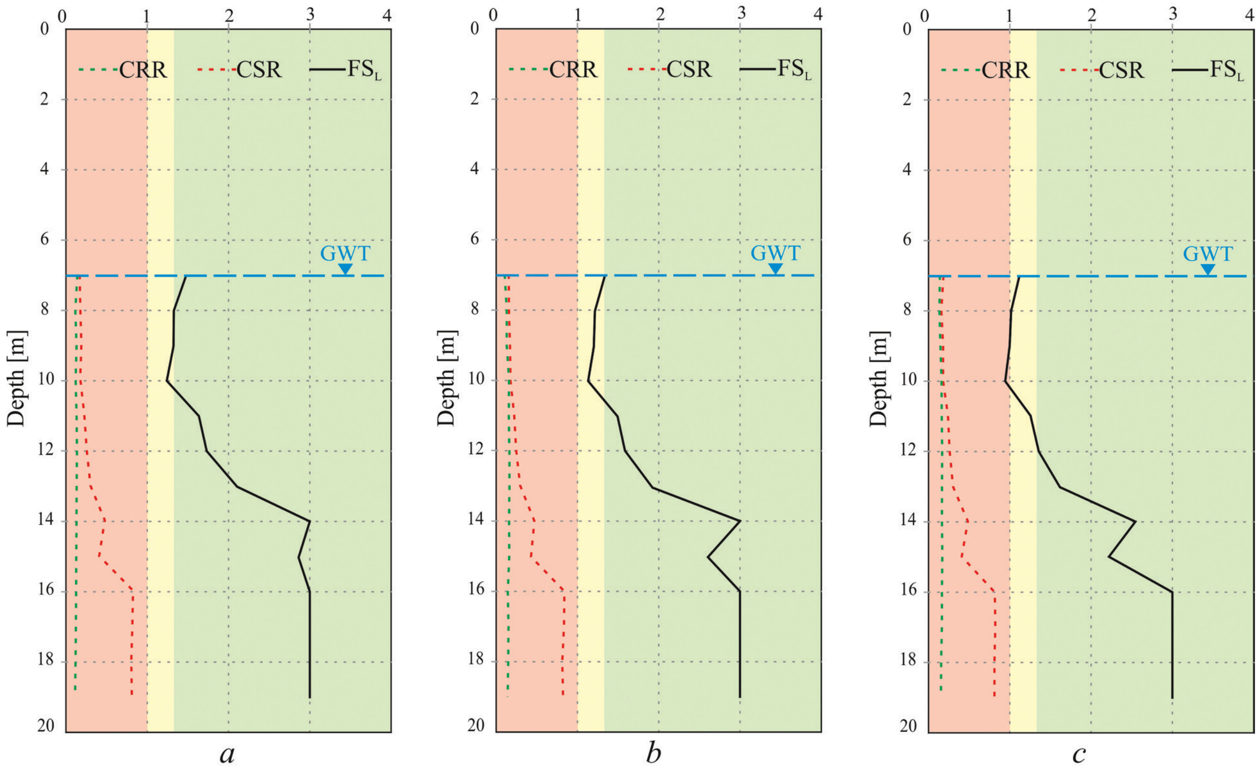

Fig. 5. Cyclic Resistance Ratio (CRR), Cyclic Stress Ratio (CSR), and factor of safety factor against liquefaction FS_L vs depth for: a_{max} = 0.20*g* (*a*); a_{max} = 0.22*g* (*b*); and a_{max} = 0.26*g* (*c*)

In Table 2, the FS_L values for the depth interval of loess layers below GWT, *i.e.*, from 7 m to 12 m, are given. It can be seen that, for seismic action with PGA of 0.20*g*, $FS_L \geq 1.25$ in the whole interval. Thus, according to EN 1998-5:2004, liquefaction will not occur at the particular seismic impact. Considering the national requirement to Eurocode 8 (EN 1998-1/NA. 2012), liquefaction is possible to occur only in the interval of 0 m to 10 m, where $FS_L < 1.33$.

At a_{max} = 0.22*g* in the depth interval of 7–10 m, *i.e.*, in the typical silty loess, FS_L is in the range of 1.13–1.34. This means that, according to the theoretical assumptions of the applied simplified procedure, liquefaction of the loess is unlikely (FS_L > 1.0), but considering some uncertainties of the field procedures it is possible to occur. In compliance with the requirements of EN 1998-5/NA 2012 Eurocode 8, at a_{max} = 0.22*g* liquefaction of the loess is possible to occur in the interval of 8–10 m, where $FS_L < 1.33$.

In the case of $a_{max} \leq 0.26g$, FS_L becomes around or lower than 1.0 in the depth interval of 8–10 m, which shows that liquefaction of the immersed silty loess is likely for the particular seismic action.

Table 2
Factors of safety FS_L for saturated loess deposits

Layer	Depth, m	FS_L at a_{max}		
		0.20*g*	0.22*g*	0.26*g*
Loess	7	1.47	1.34	1.13
	8	1.34	1.21	1.03
	9	1.33	1.2	1.02
	10	1.25	1.13	0.96
Clayey loess	11	1.64	1.49	1.26
	12	1.74	1.58	1.34

CONCLUSIONS

The liquefaction potential of loess in a ground profile representative of the first loess river terrace (T_1) within the Danube lowland areas in North Bulgaria is evaluated by the so-called simplified procedure, based on SPT for identification of potentially liquefiable soils and assessment of liquefaction hazard. The analyzed loess is a typical low-plasticity silty soil with fine content 96–98% (without clay fraction), LL = 30–32%, and PI = 10–11%.

The analysis is performed by the *NovoLiq* software applying NCEER Workshop-1997 calculation method for a seismic action with a magnitude of M=7.0 which is assumed for an equivalent magnitude for the region of Northwest Bulgaria under the impact of the Vrancea seismic zone. The safety factor against liquefaction FS_L is estimated at the respective depths in one-dimensional model of the ground profile for a free field conditions at varying peak ground accelerations a_{max} in order to assess the alteration of the FS_L. The following conclusions can be drawn:

- the analyzed saturated loess is liquefiable at $a_{\mathrm{max}} \geq 0.26g$ ($\mathrm{FS_L} < 1.0$);
- at $a_{\mathrm{max}} = 0.22g$, according to the assumptions of the applied simplified procedure, liquefaction of the loess is unlikely but possible, considering the uncertainties of the field tests. $\mathrm{FS_L}$ is in the range of 1.13–1.34, and in compliance with the requirements of the National Annex of Bulgaria to Eurocode 8 (EN 1998-1/NA. 2012) liquefaction of the saturated loess is likely to occur;
- at $a_{\mathrm{max}} \leq 0.20g$, according to EN 1998-5:2004, liquefaction of the analyzed loess cannot be expected to occur at the particular seismic impact.

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REFERENCES

Andrews, D.C.A., Martin, G.R. 2000. Criteria for liquefaction of silty soils. *Proceedings of the 12th World Conference on Earthquake Engineering, Auckland, New Zealand*, p. 312.

Berov, B., Ivanov, P., Frangov, G., Dobrev, N., Krastanov, M. 2017. Liquefaction susceptibility of quaternary deposits in Bulgaria. *Proceedings of the 17th International Multidisciplinary Scientific GeoConference SGEM 2017, 29 June–5 July, 2017, Albena, Bulgaria*, 17 (12), 499–506, https://doi.org/10.5593/sgem2017/12/S02.064.

Boulanger, R.W., Idriss, I.M. 2014. *CPT and SPT-based liquefaction triggering procedures. Report No. UCD/CGM-14/01*. University of California Davis, CA, 134 pp.

Cetin, K.O., Seed, R.B., Der Kiureghian, A., Tokimatsu, K., Harder, L.F., Kayen, R.E., Moss, R.E.S. 2004. Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential. *Journal of Geotechnical and Geoenvironmental Engineering* 130 (12), 1314–1340, https://doi.org/10.1061/(ASCE)1090-0241(2004)130:12(1314).

EN 1998-5. 2004. *Eurocode 8: Design of structures for earthquake resistance. Part 5: Foundations, retaining structures and geotechnical aspects*. BDS EN 1998-5:2005.2007 (in Bulgarian).

EN 1998-5/NA. 2012. *Eurocode 8: Design of structures for earthquake resistance. Part 5: Foundations, retaining structures and geotechnical aspects*. National annex to BDS EN 1998-5:2005/NA:2012, 14 pp. (in Bulgarian).

Evlogiev, Y. 2019. *Quaternary Geology and Geomorphology of the Danube Plain*. Geological Institute, Bulgarian Academy of Sciences, Sofia, 268 pp. (in Bulgarian).

Fardis, M., Carvalho, E., Elnashai, A., Faccioli, E., Pinto, P., Plumier, A. 2005. *Designers' Guide to EN 1998-1 and 1998-5. Eurocode 8: Design Provisions for Earthquake Resistant Structures*. Thomas Telford Publishing, London, 279 pp., https://doi.org/10.1680/dgte8.33481.

Guo, T., Prakash, S.1999. Liquefaction of silts and silt-clay mixtures*. Journal of Geotechnical and Geoenvironmental Engineering* 125 (8), 706–710, https://doi.org/10.1061/(ASCE)1090-0241(1999)125:8(706).

Idriss, I.M., Boulanger, R.W. 2008. *Soil Liquefaction During Earthquakes. Monograph MNO-12*. Earthquake Engineering Research Institute, Oakland, CA, 261 pp.

Idriss, I.M., Boulanger, R.W. 2010. *SPT-based liquefaction triggering procedures. Report No. UCD/CGM-10-02*. University of California Davis, CA, 136 pp.

Karastanev, D. 1998. Influence of moisture content on strength behavior of loess. *Proceedings of the International Symposium on Problematic soils, IS-Tohoku '98, Sendai, Japan*, 457–460.

Kishida, H. 1969. Characteristics of liquefied sands during Mino-Owari, Tohnankai and Fukui earthquakes. *Soils and Foundations* 9 (1), 75–92, https://doi.org/10.3208/sandf1960.9.75.

Pei, X., Zhang, X., Guoa, B., Wang, G., Zhang, F. 2017. Experimental case study of seismically induced loess liquefaction and landslide. *Engineering Geology* 223, 23–30, https://doi.org/10.1016/j.enggeo.2017.03.016.

Perlea, V.G., Koester, J.P., Prakash, S. 1999. How liquefiable are cohesive soils?. *Proceedings of the 2nd International Conference on Earthquake Geotechnical Engineering, Lisbon, Portugal* 2, 611–618.

Petrova, V. 2009. Engineering geological properties of loess in the town of Silistra. *Review of the Bulgarian Geological Society* 70 (1–3), 103–113 (in Bulgarian).

Seed, H.B., Idriss, I.M. 1971. Simplified procedure for evaluating soil liquefaction potential. *Journal of Soil Mechanics and Foundations Division, American Society of Civil Engineers* 97 (SM 9), 1249–1273, https://doi.org/10.1061/JSFEAQ.0001662.

Seed, H.B., Idriss, I.M., Arango, I. 1983. Evaluation of liquefaction potential using field performance data. *Journal of*

Geotechnical Engineering 109 (3), 458–482, https://doi.org/10.1061/(ASCE)0733-9410(1983)109:3(458).

Wang, W. 1979. *Some Findings in Soil Liquefaction*. Water Conservancy and Hydro-electric Power Scientific Research Institute, Beijing, China, 1–17.

Wang, W. 1984. Earthquake damages to Earth dams and levees in relation to soil liquefaction. *Proceedings of the International Conference on Case Histories in Geotechnical Engineering, University of Missouri – Rolla, MO*, 512–522.

Wang, L., Wang, Y., Wang, J., Li, L., Yuan, Z. 2004. The liquefaction potential of loess in China and its prevention. *Proceedings of the 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, August 1–6*, Paper No. 3462.

Wang, J., Yuan, Z., Li, L. 2007. Study on liquefaction of loess site. *Proceedings of the 4th International Conference on Earthquake Geotechnical Engineering, Thessaloniki, Greece, March 25–28*, https://doi.org/10.1007/s11803-002-0007-z.

Wang, G., Zhang, D., Furuya, G., Yang, J. 2014. Pore-pressure generation and fluidization in a loess landslide triggered by the 1920 Haiyuan earthquake, China: a case study. *Engineering Geology* 174, 36–45, https://doi.org/10.1016/j.enggeo.2014.03.006.

Youd, T.L., Idriss, I.M. (Eds). 1997. *Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Technical report NCEER-97-0022*. National Center for Earthquake Engineering Research, 276 pp.

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed R.B., Stokoe, K.H. 2001. Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. *Journal of Geotechnical and Geoenvironmental Engineering*, 127 (10), 817–833, http://dx.doi.org/10.1061/(ASCE)1090-0241(2001)127:10(817).