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Comparative assessment of liquefaction susceptibility of unconsolidated deposits

Évaluation comparative de la sensibilité à la liquéfaction des dépôts non consolidés

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ABSTRACT: Liquefaction may be considered one of the most dangerous physical-geological processes associated with the seismic hazard that affects almost all granular deposits as the fine ones, of low plasticity. The triggering mechanism depends on many factors that vary in space and time and may be clustered into three classes: two related to the soil properties (general settings of soil layers and geomechanical features) and one defined by earthquake characteristics. In this frame, determining the cyclic resistance of soils through in situ tests becomes an important and sensitive issue in assessing liquefaction susceptibility. This paper presents the evaluation of this seismic characteristic through several semiempirical correlations based on the most important in situ test: the Standard Penetration Test, executed on one of the most vulnerable structures which are the Holocene sedimentary alluvial deposits encountered all over the world in the proximity of every river or stream.

RÉSUMÉ: La liquéfaction peut être considérée comme l'un des processus physico-géologiques les plus dangereux associés à l'aléa sismique qui affecte presque tous les dépôts granulaires comme les plus fins, de faible plasticité. Le mécanisme de déclenchement dépend de nombreux facteurs qui varient dans l'espace et le temps et peuvent être regroupés en trois classes: deux liées aux propriétés du sol (paramètres généraux des couches de sol et caractéristiques géomécaniques de ses) et une définie par les caractéristiques sismiques. Dans ce cadre, la détermination de la résistance cyclique des sols par des essais in situ devient un enjeu important et sensible dans l'évaluation de la sensibilité à la liquéfaction. Cet article présente l'évaluation de cette caractéristique sismique à travers plusieurs corrélations semi-empiriques basées sur le test in situ le plus important: Test de Pénétration Standard, qui a été réalisé sur l'une des structures les plus vulnérables qui sont les dépôts alluviaux sédimentaires holocènes rencontrés partout dans le monde à proximité de chaque rivière ou ruisseau.

Keywords: Liquefaction, cyclic resistance, alluvial deposits.

1 INTRODUCTION

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

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

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

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

Due to the fact that the estimation of both *CRR* and *CSR* may be done through numerous semiempirical correlations with in situ test results, we chooses to perform comparative calculations based on the investigations results of three formulas of Standard Penetration Test (*SPT*), which is not only the older in situ test, but also the most widespread.

2 GENERAL FRAME OF THE SITE

The triggering mechanism of liquefaction depends on (Youd and Perkins, 1978): (i) factors related to geo-mechanical properties of soil: grain size distribution (percent of fines-*FC*, median diameter - D_{50}) and properties derived from it (hydraulic conductivity, plasticity of fines), relative density - *Dr*, frictional skills of soils; (ii) factors related to general settings of deposits: depth of the layer, thickness, overburden pressure, confining stress, nature of bed and roof layers, underground water level, and (iii) factors related to earthquake characteristics: magnitude, peak ground acceleration, intensity and duration, the distance from epicenter. In this view, alluvial layers, especially the newest Holocene ones, are the most prone to this very damaging phenomenon (Giardini et al, 2013; Ranguelov et al, 2007).

The site we refer to is located at the plain of Danube river, at short distances (less than 100km) from one of the stronger European seismic areas - namely Vrancea Seismic Zone. The specific sedimentary structure for this fluvial system consists mainly in cross-laminated fine sands, silts and mud with thin peat lenses, alternated with massive fine sandy silts and gravel bars. On selected site, these Holocene deposits in fluvial facies extend from surface to 80m up to 120m depth, and have been investigated by eight geotechnical boreholes situated at distances that do not exceed 50m one from other, in continuous rotary dry drilling system, with temporary metal casing protection, due to the fact that the water table level is very close to the terrain surface.

3 THE SUITABLE ASSESEMENT METHODS

3.1 Determination of CSR

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

$$CSR = 0{,}65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_v}{\sigma'_v}\right)r_d \qquad (2)$$

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

$$r_d = (131 - z)/131, \text{ for } z \leq 9.15\text{m};$$
$$r_d = (44 - z)/37, \text{ for } 9{,}15\text{m} \leq z \leq 23\text{m}; \quad (3)$$
$$r_d = (93 - z)/125, \text{ for } 23\text{m} \leq z \leq 30\text{m};$$
$$r_d = 0{,}50, \text{ for } z \geq 30\text{m}$$

3.2 Determination of CRR

The calculations of CRR were performed through three deterministic methods, all based on SPT resistance, expressed as functions of corrected and normalized values $(N_1)_{60}$ and of clean sand corrected N-value $(N_1)_{60cs}$.

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

$$CRR_{7.5} = \frac{1}{34-(N_1)_{60cs}} + \frac{(N_1)_{60cs}}{135} + \frac{50}{[10(N_1)_{60cs}+45]^2} - \frac{1}{200} \qquad (4)$$

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

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \qquad (5)$$

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

(II) In the second set of relations, Idriss and Boulanger (2004) express $CRR_{7.5}$ from equation (6):

$$CRR_{7.5} = exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right) \qquad (6)$$

in which $(N_1)_{60cs}$ has been calculated based on Idriss and Boulanger (2008), formulas (7).

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$$
$$\Delta(N_1)_{60} = exp\left(1.63 + \frac{9.7}{FC+0.01} - \left(\frac{15.7}{FC+0.01}\right)^2\right) \qquad (7)$$

(III) Finally, Japanese Bridge Code-JR (1990) attests that $CRR_{7.5}$ is affected by the median diameter of grain size distribution curve (D_{50}) as follows for $0.05mm \leq D_{50} \leq 0.6mm$ (8):

$$CRR_{7.5} = 0.0882\sqrt{\frac{(N_1)_{60cs}}{\sigma'_v+0.7}} + 0.255log\left(\frac{0.35}{D_{50}}\right) + R_3 \quad (8)$$

in which dimensionless coefficient $R_3=0$ for $FC<40\%$ and $R_3=0.004FC-0.16$ for $FC \geq 40\%$.

In this third calculation, $(N_1)_{60cs}$ has been used based on Idriss and Boulanger (2008) formulas.

4 RESULTS

In the first instance, we performed a single calculation of CSR as described in paragraph 3.1., considering a value a_{max}=0.30g, according to Romanian Seismic Code, for the depths where simultaneously grainsize distribution tests and SPT tests has been made.

We continued by applying the three variants of calculation of $CRR_{7.5}$ as described in paragraph 3.2. After normalization of SPT values, with the aim of calculation of the equivalent clean-sand corrected blow count $(N_1)_{60cs}$, we applied the relations (5) and (7), which proved to offer similar results on the first 10m, and slightly to significant different from 10m to 30m depth.

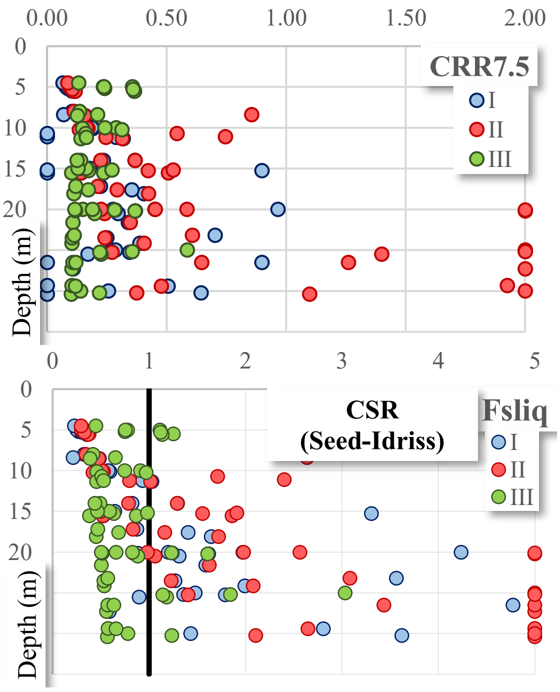

Figure 1. Comparative assessment of variation in depth of $CRR_{7.5}$ (up); factor safety against liquefaction according to Seed-Idriss (down).

In the following, we applied relations (4), (6) and (8), considering an event of magnitude 7.5, for which consequently the magnitude scaling factor is $MSF=1$, and a value of $K_\alpha=1$ for level ground. As for overburden stress correction factor K_s, we applied recommendations of Youd et.al (1996). The results are graphically presented in Figure 1.

In second instance, we recalculate CSR according to Eurocode 8, Part 5 and reevaluate the main parameter Fsliq through relations (4), (6) and (8), in the same conditions. The results are graphically presented in Figure 2, in which the differences ΔFsliq is given by the formula:

$$\Delta Fs_{liq} = Fs_{liq}^{Seed-Idriss} - Fs_{liq}^{Eurocod\,8} \quad (9)$$

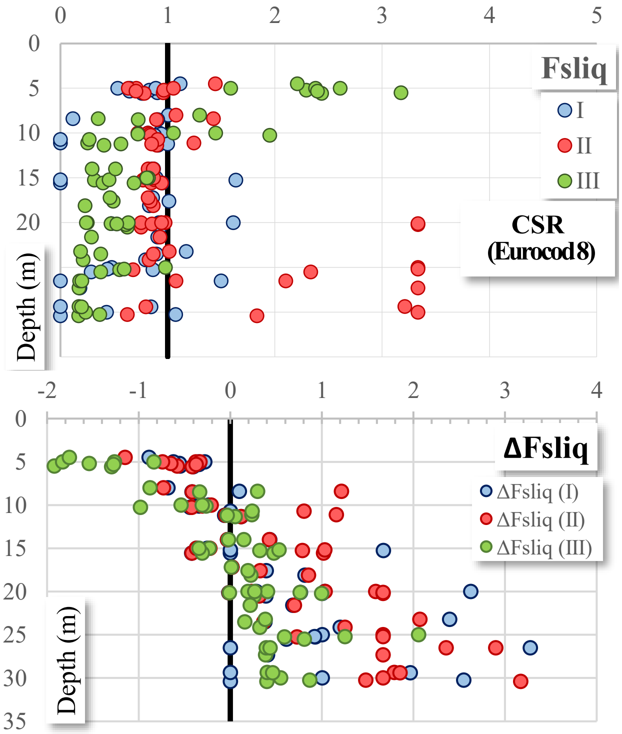

Figure 2. Factor safety against liquefaction according to Eurocod 8 (up); differences ΔFsliq between calculations (down).

5 CONCLUSIONS

A. Regarding the comparative calculation of the cyclic resistance ratio (CRR):

- as is presented in Figure 2 (up), the procedure II, Idriss and Boulanger (2004), provide the most overrated values compared to the other two set of results, while the Japanese Bridge Code-JRA (1990) - procedure III, offers the most conservative results;

- these ranges of values conducted at a safety profile against liquefaction mainly secure bellow 15m depth, according to procedures (I) and (II), while relations (III) extend the unsafety of the ground up to 30m depth;
- consequently, authors consider that procedures (I) or (II) are more suitable for shallow foundations, and procedure (III) is more appropriate for deep foundations.

B. Regarding the comparative calculation of the cyclic shear stress (CSR):

- the range of values resulting from Seed and Idriss formulas (Eq. 2) is strongly decreasing in depth and in consequence, the values Fsliq will directly increase in the same direction;
- on the other hand, the evaluation of CSR according to Eurocod 8 is influenced primarly by the fines percents and subservient by depth; this dependency makes that for the same set of data, the values Fsliq will decrease in the depth for most calculations;
- differences between these two procedures of calculation of CSR (Eq. 9), drive to large differences in term of factor of safety, wich varies $-2< \Delta Fsliq<3$, with prevalent negative values recorded above the depth of 15m (Figure 2 down).

REFERENCES

Anwar, A., Ahmad, S., Jamal, Y. and Khan, M.Z. (2016). Assessment of Liquefaction Potential of Soil Using Multi-Linear Regression Modeling, *International Journal of Civil Engineering and Technology*, 7 (1), 373-415.

Cetin K.O., Seed R.B., Der Kiureghian A., Tokimatsu K., Harder L.F. Jr, Kayen R.E. and Moss R.E.S. (2004). SPT-based probabilistic and deterministic assessment of seismic soil liquefaction potential, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 130 (12), 1314-1340. doi.org/10.1061/(ASCE)10900241(2004)130:12(1314).

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

Giardini D., Woessner J., Danciu L., Cotton F., Crowley H., Grünthal G., Pinho R., Valensise G., Akkar S., Arvidsson R., Basili R., Cameelbeck T., Campos-Costa A., Douglas J., Demircioglu M. B., Erdik M., Fonseca J., Glavatovic B., Lindholm C., Makropoulos K., Meletti C., Musson R., Pitilakis K., Rovida A., Sesetyan K., Stromeyer D. and Stucchi M. (2013). *Seismic Hazard Harmonization in Europe*: Online Data Resource, doi:10.12686/SED-00000001-SHARE.

Idriss, I. M. (1999). An update to the Seed-Idriss simplified procedure for evaluating liquefaction potential in *Proceedings, TRB Workshop on New Approaches to Liquefaction,* Publication No. FHWA-RD-99-165, Federal Highway Administration, January.

Idriss, I.M. and Boulanger, R.W. (2004). Semi-empirical procedures for evaluating liquefaction potential during earthquakes. *Proceedings of 11th International conference on soil dynamics and earthquake engineering, and 3rd International conference on earthquake geotechnical engineering*, vol. 1. Stallion Press. 32–56.

Idriss, I. M., and Boulanger, R. W. (2008). *Soil liquefaction during earthquakes*. Monograph MNO-12, Earthquake Engineering Research Institute, Oakland, CA, 261 pp.

Ishihara, K. (1993). Liquefaction and Flow Failure during Earthquake. *Géotechnique*, 43, 351-415. doi.org/10.1680/geot.1993.43.3.351.

JRA (1990). *Specification for Highway Bridges*: Part V- Seismic Design. Japan Road Association, Tokyo.

Liao, S.S.C. and Whitman, R.V. (1986). Overburden correction factors for SPT in sand. *Journal of Geotechnical Engineering*, 112, (3), 373-377. doi.org/10.1061/(ASCE)07339410(1986)112:3(37).

Ranguelov, B., Jelínek, R., Wood, M. and Hervás J., (2007). Risk Mapping of Earthquakes in New Member States. *JRC Scientifc and Technical Reports*, 38pp, ISSN 1018-5593.

Seed, H. B., and Idriss, I. M. (1971). Simplified Procedure for Evaluating Soil Liquefaction Potential, *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 97, No SM9, Proc. Paper 8371, September 1971, 1249-1273.

Seed, H. B., Tokimatsu, K., Harder, L. F. and Chung, R. M. (1984). The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations, *Earthquake Engineering Research Center Report No. UCB/EERC-84/15*, Univ. of California at Berkeley.

Seed, R.B. and Harder, L.F. (1990). SPT-based analysis of cyclic pore pressure generation and undrained residual strength. In *Proceedings of the H. Bolton Seed Memorial Symposium*, Berkeley, Calif. Edited by J.M. Duncan. BiTech Pub- lishers Ltd., Vancouver. Vol. 2. 351-376.

Youd, T. L. and Perkins, D.M. (1978). Mapping liquefaction-induced ground failure potential. *Journal of the Geotechnical Engineering Division.* ASCE,104, GT4, Proc. Paper 13659, April, 1978, 433-446.

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. and 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, 297-313. dx.doi.org/ 10.1061/ (ASCE)1090-0241(2001)127:4(297).