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LIQUEFACTION POTENTIAL EVALUATION AT RIPOSTO HARBOUR (ITALY)

R. Cerra[1], A. Scotto di Santolo[2], S. Grasso[3] & M.S.V. Sammito[4]

[1] Telematic University Pegaso, Italy

[2] Telematic University Pegaso, Italy, anna.scottodisantolo@unipegaso.it

[3] University of Catania, Catania, Italy

[4] "Kore" University of Enna, Enna, Italy

Abstract: *According to the frequency and the importance of the seismic effects suffered in past times, Eastern Sicily is considered one of the highest seismic risk areas in Italy. The area to the south of Volcano Etna, on the east of the Ibleo Maltese escarpment, known as Iblean Area, is therefore a seismogenic area. The city of Riposto in South-Eastern Sicily has been affected by several destructive earthquakes of about magnitude 7.0+ in past times; a repetition of events with similar characteristics would provide the additional risk of a damaging tsunami, as well as liquefaction phenomena around the coast. Seismic liquefaction phenomena were reported by historical sources following the 1693 and 1818 earthquakes. In situ investigations of sandy harbour soil were carried out in order to determine the soil profile and the geotechnical characteristics for the site under consideration. One of the most common parameters for estimating soil resistance to liquefaction is the number of blows N obtained from standard penetration tests (SPT). Indeed, the* N_{SPT} *value, not only reflects the soil relative density and the soil fabric, but also allows to estimate soil shear strength in undrained conditions. In this paper, the susceptibility of the site to seismic-induced liquefaction has been assessed comparing the cyclic soil resistance to the cyclic shear stresses due to ground motion using blow counts from the SPT tests performed in the area under consideration.*

1. Introduction

Earthquake-induced soil liquefaction is an important research field in geotechnical seismic engineering. Liquefaction effects in some major earthquakes (Anchorage, 1964; Niigata, 1964; Kobe, 1995; Kocaeli, 1999; Christchurch, 2010-2011; Tohoku 2011; Palu, 2018; among others) caused severe damage to the built environment. An accurate assessment of the liquefaction potential for susceptible soils can prevent major casualties and economic losses caused by damage (Lirer et al. 2020; Yang et al. 2023).

The commonly used "Simplified Procedure" proposed by Seed and Idriss (1971) evaluates the triggering of liquefaction by comparing the resisting capacity of the soil to liquefy and the design seismic action at a specific location in depth (Ferreira et al. 2020). In situ tests widely used to evaluate the liquefaction resistance of soil are the Standard Penetration Tests (SPT) (Boulanger and Idriss 2014; Castelli et al. 2016) and Cone Penetration Tests (CPT) (Boulanger and Idriss 2016). SDMT tests (Grasso and Maugeri 2008; Grasso et al. 2021) and Shear Wave Velocity (V_S) measurements (Andrus and Stokoe 2000; Grasso et al. 2006) can also be employed to assess the liquefaction resistance of soil.

In this work, laboratory and in situ investigations have been performed to define the subsoil model and the geotechnical characteristics of the sandy soil in the city of Riposto (Italy). The Val di Noto earthquake of January, 11, 1693, has been chosen as scenario earthquake. It is considered one of the biggest seismic events occurred in Italy with an estimated magnitude of 7.3 on the moment magnitude scale. Ground response analyses have been carried out by the 1-D linear equivalent code STRATA (Kottke and Rathje 2008). Results of site response analyses have been employed to evaluate the liquefaction potential of the investigated area. The "Simplified Procedure" has been applied to SPT date available in the Riposto Harbour area.

2. Site Response Analyses

2.1. Definition of the subsoil model

The municipality of Riposto is located on the eastern Sicily (Italy) (Figure 1). The municipal area, included in the province of Catania, extends for 12.96 km^2. It is one of the most seismically active area of Italy that suffered the destructive earthquakes of 1169, 1693, 1818 and 1990 (Ferraro et al. 2018).

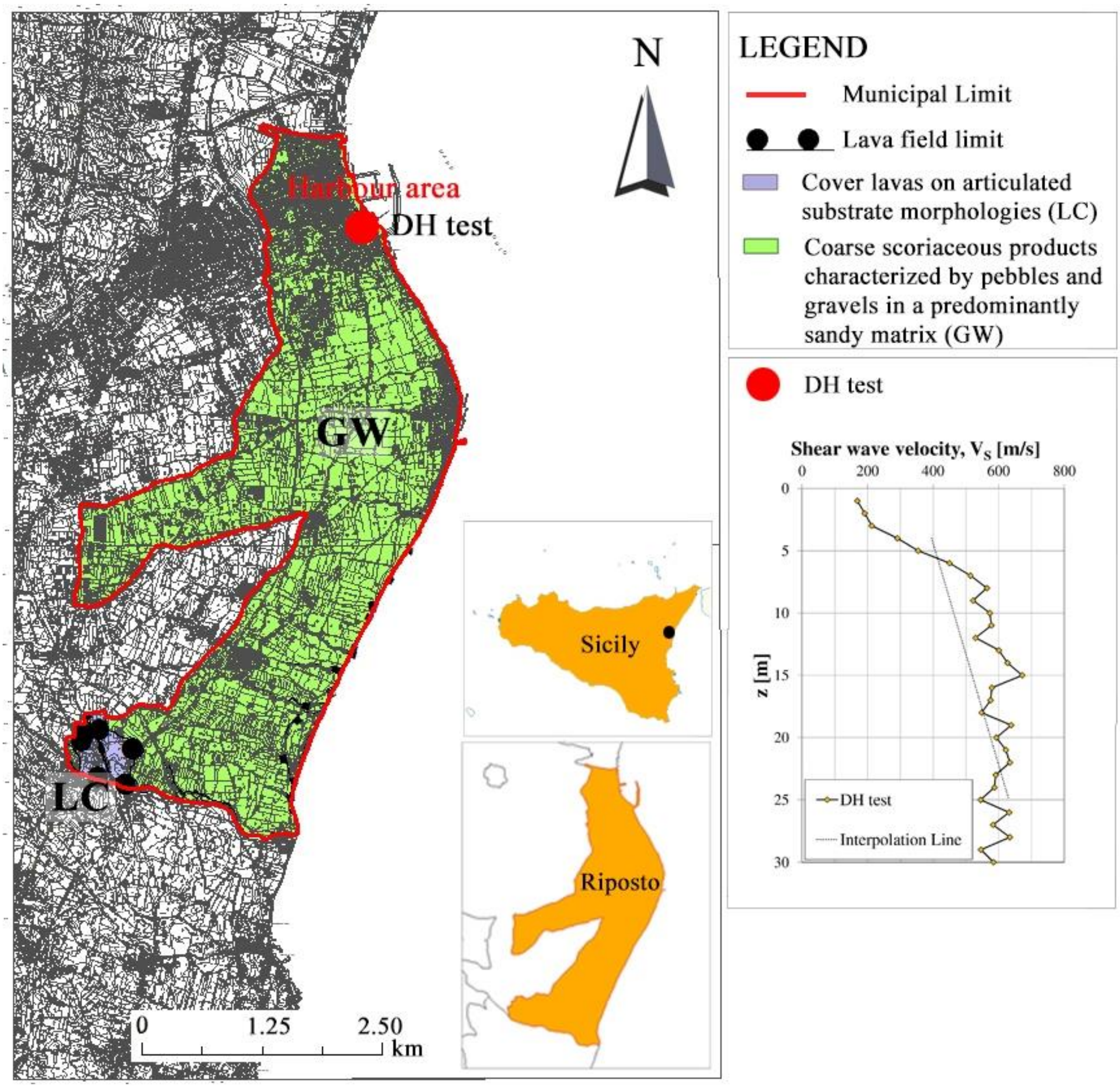
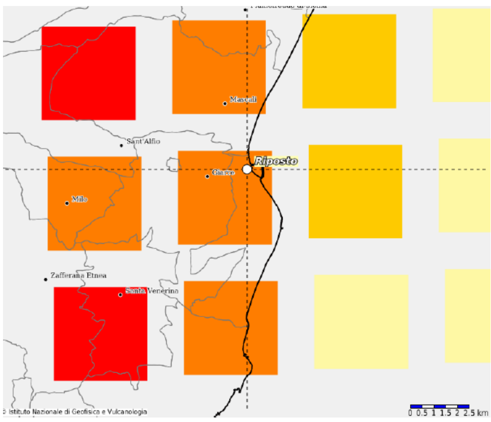


Figure 1. Geological map of the study area and shear wave velocities from the DH test.

In order to define the subsoil geometry and the seismic characterization of the area under investigation, the following laboratory and in situ investigations were performed: n. 11 Boreholes, n. 11 SPT tests, n. 1 Down Hole Tests (DH), n. 21 Horizontal to Vertical Spectral Ratio Tests (HVSR) and n. 6 Direct Shear Tests (DST). The study area is mainly composed of the “Chiancone” formation (lithological unit GW=coarse scoriaceous products characterized by pebbles and gravels in a predominantly sandy matrix) and, in a small zone, cover lithoid (lithological unit LC=cover lavas on articulated substrate morphologies) overlays the “Chiancone” (Cavallaro et al. 2022; Castelli et al. 2023). The subsoil model has been defined from the V_S profile obtained from the Down-Hole test performed in the Harbour area. The depth of the bedrock ($z_{bedrock}$=39 m) has been derived from the linear interpolation of the V_S profile (Figure 1).

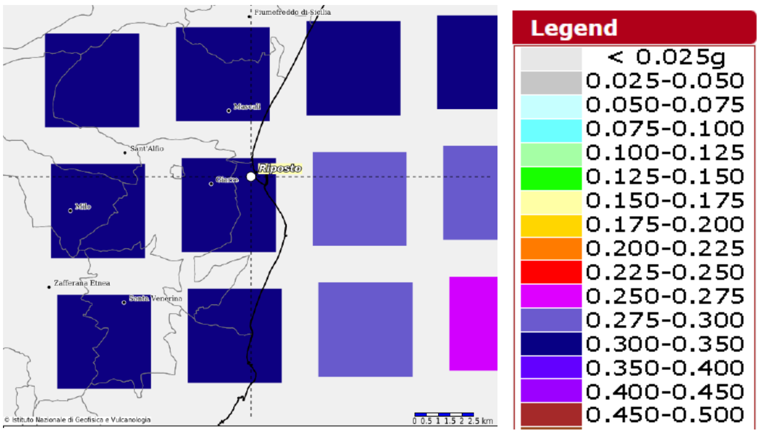
2.2. Seismic input

The 1D numerical analyses were carried out through STRATA (Kottke and Rathje 2008), operating in the frequency domain, based on the equivalent-linear visco-elastic approach to model the cyclic soil behaviour. In order to describe the decay of the shear modulus and the increase in damping as a function of the level of deformation, it has been necessary to refer to dynamic curves from the literature (Castelli et al. 2018). A linear visco-elastic behaviour has been assigned to the seismic bedrock and the outcropping condition has been applied at the base of the soil column. A synthetic seismogram, evaluated assuming the source to be along the Hyblean-Maltese fault and generating the 1693 seismic ground motion scenario (Grasso et al. 2005), has been chosen as seismic input in this study. Based on the interactive seismic hazard maps of the city of Riposto (Meletti and Montaldo 2007) (Figure 2), the synthetic seismogram has been scaled to the maximum PGA of 0.225 g (corresponding to a return period, T_R, of 475 years), to the maximum PGA of 0.275 g (T_R=975 years) and to the maximum PGA of 0.350 g (T_R=2475 years) by entering the scale factors in the STRATA code (Figure 3).



(a)

(b)



(c)

Figure 2. Interactive seismic hazard maps of the city of Riposto: (a) 10% probability of exceedance in 50 years (T_R=475 years); (b) 5% probability of exceedance in 50 years (T_R=975 years); (c) 2% probability of exceedance in 50 years (T_R=2475 years).

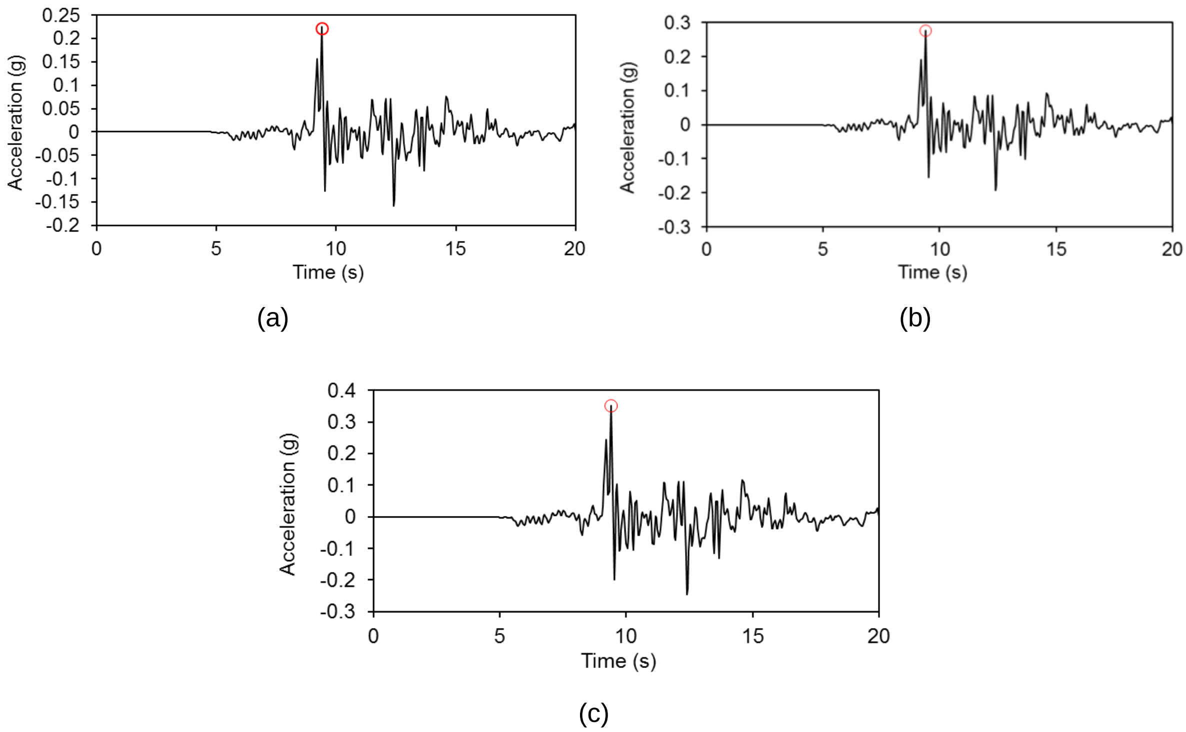


Figure 3. Scaled synthetic seismograms of 1693 to the maximum PGA of (a) 0.225 g (T_R=475 years) (b) 0.275 g (T_R=975 years) and (c) 0.350 g (T_R=2475 years).

2.3. Analyses of the results

Using the time histories reported in Figure 3, the maximum accelerations with depth, the response spectra and the amplification functions concerning the investigated site have been deduced.

Figure 4 shows the results in terms of maximum accelerations with depth for the 475, 975 and 2475 return periods. Values of the amplification ratio, R, calculated as the ratio between output and input PGA, are displayed in Table 1.

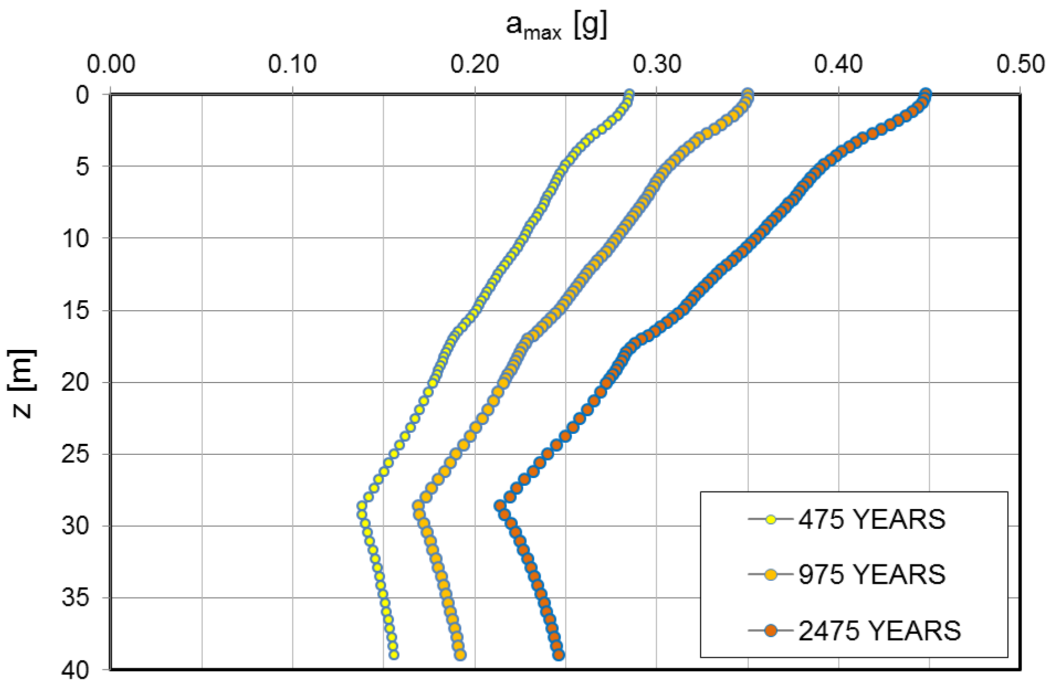
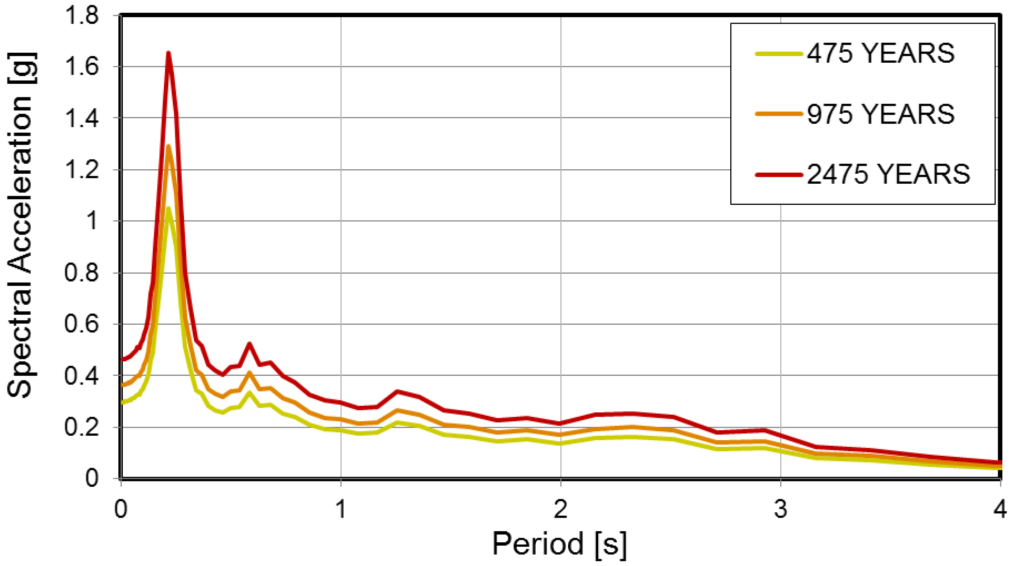


Figure 4. Maximum accelerations with depth obtained by numerical analyses using the synthetic seismogram of 1693.

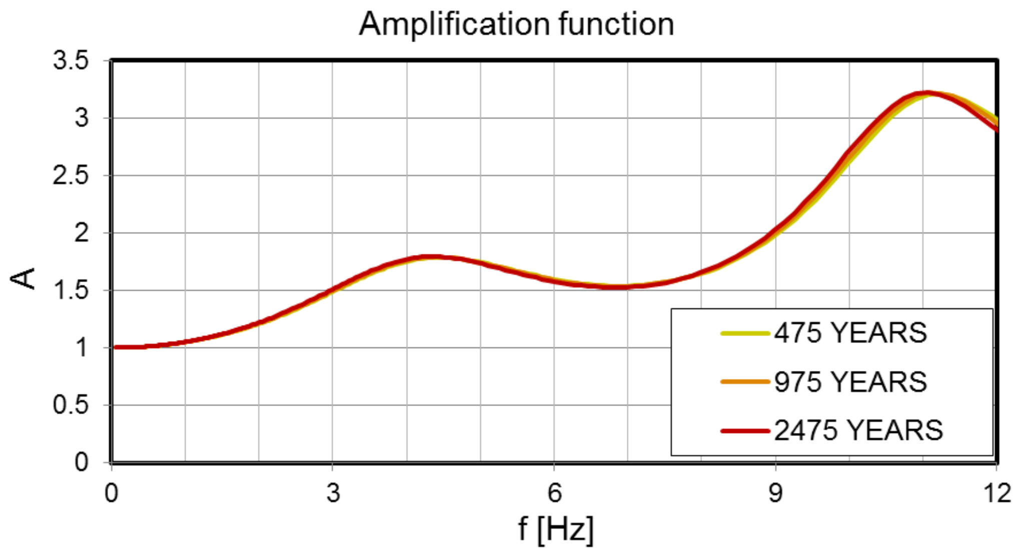
Table 1. Values of the surface maximum accelerations and amplification ratio obtained by numerical analyses using the synthetic seismogram of 1693.

	T_R=475 years	**T_R=975 years**	**T_R=2475 years**
PGA_{input}	0.225 g	0.275 g	0.350 g
PGA_{output}	0.258 g	0.351 g	0.448 g
R= PGA_{output}/PGA_{input}	1.267	1.275	1.280

Figure 5 (a) shows the response spectra obtained at the surface by setting a structural damping of 5%. It is possible to observe that the maximum spectral accelerations of 1.07 g (T_R=475 years), 1.26 g (T_R=975 years) and 1.65 g (T_R=2475 years) are derived at T = 0.21 s. Finally, Figure 5 (b) reports the amplification functions, A(f), evaluated as the ratio between the Fourier spectrum at the surface level and the Fourier spectrum of the input motion applied at the base of the model.



(a)



(b)

Figure 5. (a) Response spectra and (b) amplification functions obtained by numerical modelling using the synthetic seismogram of 1693.

3. SPT procedure for evaluating soil liquefaction

The simplified procedure for evaluating soil liquefaction potential requires the estimation of two variables: the seismic demand on a soil layer, expressed in terms of cyclic stress ratio (CSR) and the capacity of the soil to resist liquefaction, expressed in terms of cyclic resistant ratio (CRR) (Youd et al. 2001). Therefore, the evaluation of the susceptibility of a site to seismic-induced liquefaction can be assessed comparing the CSR to the CRR.

The cyclic stress ratio CSR has been calculated by the following equation (Boulanger and Idriss 2014):

$$CSR = \frac{\tau_{av}}{\sigma'_{v0}} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right)r_d \quad (1)$$

where τ_{av}=average cyclic shear stress, a_{max}=peak horizontal acceleration at the ground surface (Table 1), g=acceleration of gravity, σ_{v0} and σ_{v0}'=total and effective overburden stresses, r_d=stress reduction coefficient.

The r_d values that describe the ratio of cyclic stress for a flexible soil column to the cyclic stress for a rigid soil column (Cetin and Seed 2004) have been derived by the following equations:

$$r_d = 1.0 - 0.00765z \qquad for\ z \leq 9.15m \quad (2)$$

$$r_d = 1.174 - 0.0267z \quad for\ 9.15m < z \leq 20m \quad (3)$$

$$r_d = 0.744 - 0.008z \qquad for\ 23m < z \leq 30m \quad (4)$$

where z=depth below ground surface in meters. The Equations (2) and (3) were proposed by Liao and Whitman (1986) and the Equation (4) was added by Robertson and Wride (1997).

In this work, the results obtained from the site response analyses in terms of maximum accelerations at the surface (Table 1) have been used to evaluate the CSR (Figure 6). Thus, the values of 0.258 g (T_R=475 years), 0.351 g (T_R=975 years) and 0.448 g (T_R=2475 years) have been chosen for the peak horizontal acceleration at the ground surface.

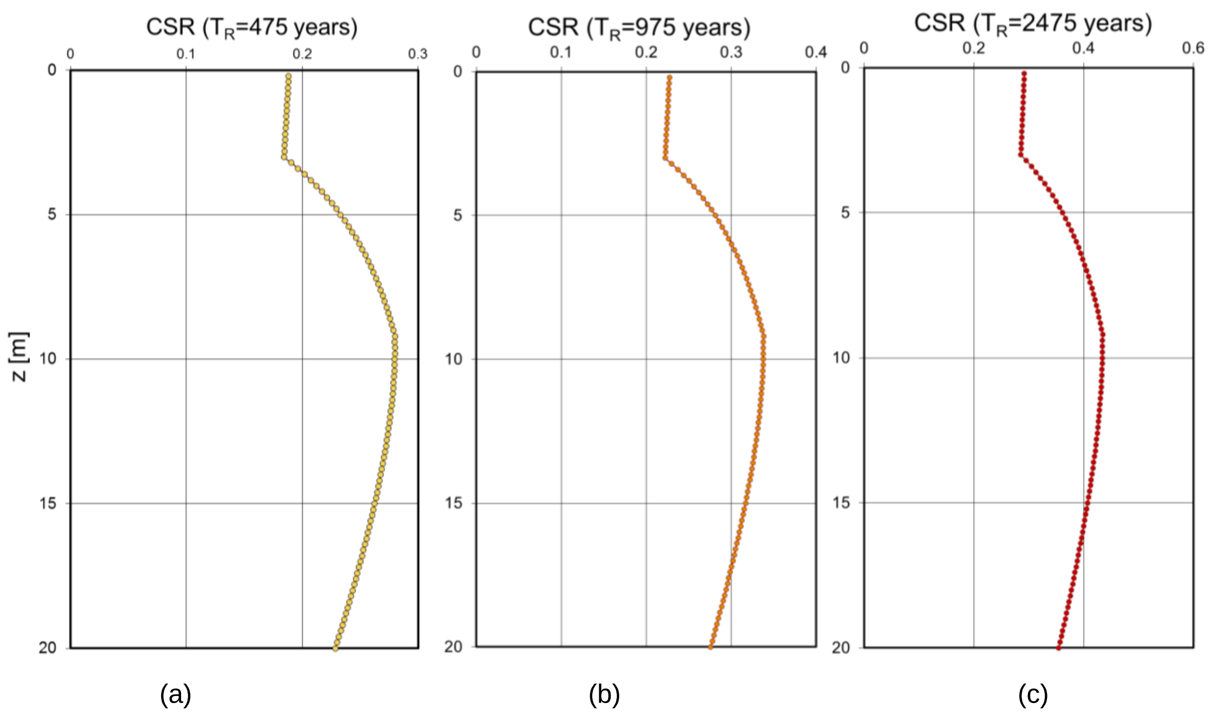


Figure 6. CSR profiles obtained from Equation (1) for: (a) T_R=475 years, (b) T_R=975 years and (c) T_R=2475 years.

CRR have been evaluated from the SPT tests performed in the Harbour area (Figure 6) by the following expression (Robertson and Wride 1997):

$$CRR_{7.5} = \frac{a + cx + ex^2 + gx^3}{1 + bx + dx^2 + fx^3 + hx^4} \quad (5)$$

where $CRR_{7.5}$ is the cyclic resistance ratio for magnitude 7.5 earthquakes; x is the corrected SPT blow counts, $(N_1)_{60}$; a=0.048; b=-0.1248; c=-0.004721; d=0.009578; e=0.0006136; f=-0.0003285; g=-0.00001673 and h=0.000003714.

The values of $CRR_{7.5}$ have been scaled to a magnitude of M=7.3 of the 1693 scenario earthquake by the following expression (Youd et al. 2001):

$$MSF = \frac{10^{2.24}}{M^{2.56}} \quad (6)$$

Figure 7. Location of SPT tests in the Harbour area.

One way to quantify the potential for liquefaction is through the computation of the liquefaction resistance factor (F_{SL}) profile given by the ratio of the cyclic resistance ratio CRR(z) to cyclic shear stress ratio CSR(z), where z is the depth of the deposit. Then it is possible to evaluate the liquefaction potential index (LPI). The LPI, proposed by Iwasaki et al. (1978), provides a depth-weighted index of the potential for triggering of liquefaction at a site:

$$LPI = \int_0^{20} F(z)w(z)dz \quad (7)$$

where:

$$\mathrm{F(z)} = 0 \text{ for } \mathrm{F_{SL}} \geq 1 \quad (8)$$

$$\mathrm{F(z)} = 1 - \mathrm{F_{SL}} \text{ for } \mathrm{F_{SL}} < 1 \quad (9)$$

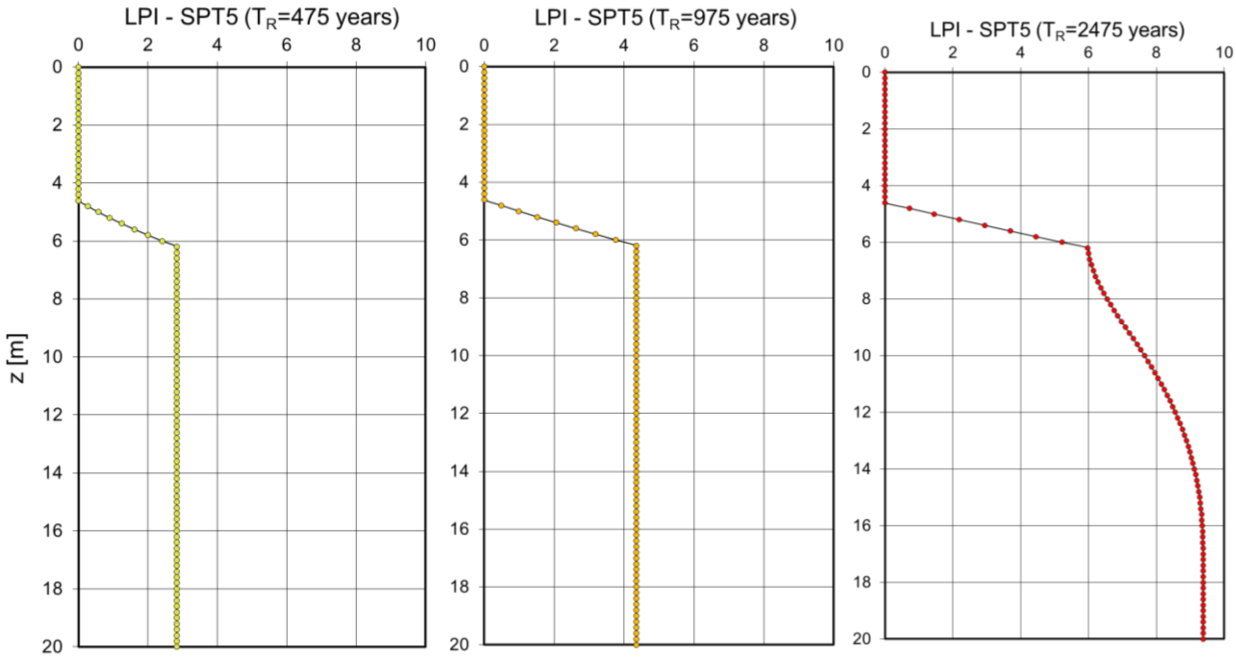
$$\mathrm{w(z)} = 10 - 0.5\mathrm{z} \quad (10)$$

According to Iwasaki et al. (1982), the level of liquefaction severity with respect to LPI is given in Table 2.

Table 2. Level of liquefaction severity (Iwasaki et al. 1982).

LPI	Level of Liquefaction Severity
0	Very low
0<LPI ≤5	Low
5< LPI ≤15	High
LPI >15	Very high

The LPI values obtained from SPT5, as example, are reported in Figure 8 for the 475, 975 and 2475 return periods.



(a) (b) (c)

Figure 8. LPI values obtained from SPT5 for: (a) T_R=475 years, (b) T_R=975 years and (c) T_R=2475 years.

Based on the results obtained by the SPT5, the LPI value is higher than 5 (high liquefaction potential) at the depth of about 6 meters for T_R=2475 years; while the LPI values for T_R=975 years and T_R=2475 years show a low liquefaction potential. Similar results have been derived from other SPT tests.

4. Conclusions

In this work, some information concerning the geotechnical characterisation for soil liquefaction evaluation of the sandy soil in the city of Riposto (Italy) have been delineated. Local site response analyses have been performed by the 1D linear equivalent code STRATA (Kottke and Rathje 2008) using a synthetic seismogram of the 1693 earthquake.

Results of site response analyses in terms of maximum accelerations at the surface for the 475, 975 and 2475 return periods of the scenario earthquake have been used for the evaluation of the CSR in the framework of the simplified procedure originally proposed by Seed and Idriss (1971). The correlation to CRR is based on the corrected penetration resistance derived by SPT data.

The liquefaction potential index (LPI) (Iwasaki et al. 1982) values show a low liquefaction potential for the 475 and 975 return periods and a high liquefaction potential for the 2475 return period results demonstrating that the maximum accelerations at the surface is an important parameter in the evaluation of liquefaction potential.

5. Acknowledgments

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