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Nonlinear numerical simulation of the soil seismic response to the 2012 Mw 5.9 Emilia earthquake considering the variability of the water table position

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This research is focused on the case study of San Carlo village (Emilia Romagna, Italy), struck by the 20 May 2012 Mw 5.9 Emilia earthquake that caused severe damage due to widely observed soil liquefaction. More in particular, it is investigated the influence on nonlinearity effects of the variability of the water table depth, due to seasonal fluctuation.

The one-directional propagation of a three-component seismic wave (1D-3C approach), in a multilayered soil profile, is simulated using a finite element model and an elasto-plastic constitutive behavior with hardening for the soil (Iwan’s model). The nonlinearity is described by the normalized shear modulus decay curve obtained by resonant column tests. The shear modulus is corrected during the process (Iai’s model) to consider the cyclic mobility and dilatancy of sands, depending on the actual average effective stress and the friction and dilatancy angles obtained from cyclic consolidated undrained triaxial tests.

Profiles with depth of maximum excess pore water pressure, horizontal motion and shear strain and stress are obtained in the case of effective stress analysis, for an average position of the water table depth and for a variation of ±1 m, and then compared with a total stress analysis. The variability with the water table depth of soil profile response to seismic loading is observed also in terms of hysteresis loops, time histories of the ground motion and excess pore water pressure in the liquefiable soil layers prone to cyclic mobility process.

Keywords: saturated soil, liquefaction front, wave propagation, Finite Element Method, Emilia earthquake.
INTRODUCTION

During the 20 May 2012 Mw 5.9 Emilia earthquake (Italy), liquefaction phenomena have been observed (Chini et al., 2015; Emergeo Working Group, 2013). Several studies (Facciorusso et al., 2014, 2015; Vannucchi et al., 2012; Papathanassiou et al., 2015) demonstrate that predisposing conditions to soil liquefaction could be recognized in several sites (including the villages of San Carlo, Mirabello and Sant’Agostino) where such an effect was induced by the 2012 Emilia earthquake. Focusing on this case study, the impact of water table depth on the earthquake-induced effects in soil columns is here analyzed.

Effects due to co-seismic water level changes, inducing soil liquefaction, are observed and discussed by Wang et al. (2003) while Wayne and Dohering (2006) observe liquefaction induced by measured water level changes after detonation in underground of chemical and nuclear explosives. Some authors (Nishikawi et al., 2009; Yasuda and Ishikawa, 2015) discuss the possible role of water table lowering for the soil improvement against co-seismic liquefaction occurrence and building damage. Moreover, a co-relations of shallow groundwater levels with the liquefaction occurrence is proposed by Hartantyo et al., 2014 for the May 2006 Earthquake at Yogyakarta (Indonesia). Some analytical evaluations are performed to consider the effect of water level on analytical indexes for liquefaction susceptibility as well as on induced post-seismic settlements (Chung and Rogers, 2013).

In this research, a numerical modeling is developed to simulate the seismic response of the soil columns through effective stress analysis (ESA). A vertical wave propagation model is adopted under the assumption of horizontally extended soil. The one-directional (1D) wave propagation modeling, compared with a three-dimensional (3D) one, reduces modeling difficulties and computation time, and guarantees a reliable geotechnical model, easy to characterize with
The Iwan’s elasto-plastic model (Iwan, 1967; Joyner, 1975; Joyner and Chen, 1975) is adopted to represent the 3D nonlinear behavior of soil. Its main feature is the faithful reproduction of nonlinear and hysteretic behavior of soils under cyclic loadings, with a reduced number of parameters characterizing the soil properties. The model is calibrated using the elastic moduli in shear and compression and the shear modulus decay curve is employed to deduce the size of the yield surface.

The correction of the shear modulus proposed by Iai et al. (1990a,b) is employed for saturated cohesionless soil layers to consider the cyclic mobility and dilatancy of sands. Liquefaction front parameters are calibrated by a trial-and-error procedure to best reproduce the curves obtained by cyclic consolidated undrained triaxial (CTX) tests, that represent the deviatoric strain amplitude and normalized excess pore water pressure with respect to the number of cyclic loading. Bonilla et al. (2005) propose the Iwan’s hysteretic model combined with the Iai’s liquefaction front model. The model is applied to a one-directional one-component seismic wave propagation, in a finite difference formulation. Oral et al. (2017) use the extension of Iai’s shear modulus correction to a multiaxial stress state, induced by a 3C excitation. The model is adopted in a spectral element formulation, for a three-component seismic loading propagating in a 1D soil profile. Santisi d’Avila et al. (2018) discuss the 3D Iwan-Iai model and compare the behavior of dense and loose sands under one- and three-component loading. They also validate the one-
directional three-component (1D-3C) wave propagation model in liquefiable soils, for the case of KSRH10 Japanese soil profile, whose geotechnical data are provided by the reports of PRENOLIN benchmark.

Stratigraphy and geotechnical parameters of three soil profiles in San Carlo village (Emilia Romagna, Italy) are accurately identified based on available data and specifically performed borehole investigations. The simulation of their response to the 20 May 2012 Mw 5.9 Emilia earthquake is initially performed by considering an average depth of the water table. In the following, the influence of the water table depth variability on the seismic response of the investigated soil profiles is accounted for and the obtained results are discussed, in terms of ground motion at the surface and profiles with depth of maximum stress, strain and soil motion. The case study is of particular interest for the Italian National Civil Protection since it pointed out emergency conditions induced by liquefaction effects. Moreover, due to the similarity of geological setting of the area felt by the 2012 Emilia earthquake with large zones of the Pianura Padana plain (northern Italy) this case study exemplifies conditions that could be somewhere else expected.

Due to the co-seismic nature of the nonlinear effects related to soil liquefaction, this study contributes to depict quantitative provisional scenarios for risk mitigation by identifying geological setting prone to liquefaction and supporting the outline of unstable zones in the framework of Seismic Microzonation studies (DPC-CRP, 2008).

**CASE STUDY**

On May 2012 a seismic sequence hit the river Po valley plain in Northern Italy, with two mainshocks of magnitude close to 6 (Scognamiglio et al., 2012) which triggered several ground
effects mainly represented by liquefaction (Fig. 1a,b) in all its variegated phenomena (i.e. sand boils crack fissures, lateral spreading, settlements). Such effects significantly contributed to increase the structural damages due to the seismic shaking (Fig. 1c,d).

The investigated site was shaken by earthquakes of similar magnitude which produced similar ground effects in the past (Martino et al., 2014), as reported in the Italian catalogue of earthquake-induced ground effects (CEDIT, see Data and Resources). The seismic sequence, well described in Scognamiglio et al. (2012), consisted of two mainshocks, the first of which ($M_w$ 5.9 on May 20th, 2012) was responsible for the liquefaction effects at the investigated site (16 km far from the epicentre). The inferred peak ground acceleration according to the automated shakemaps (INGV, see Data and Resources) is about 0.32g.

After the seismic sequence, many investigations are carried out, both on site and in laboratory, in order to explore the soil susceptibility to undergo failure. Some studies are already published attempting to explain the triggering mechanisms of the observed phenomena (Vannucchi et al., 2012; Emergeo Working Group, 2013; Facciorusso et al., 2014; Papathanassiou et al., 2015; Chini et al., 2015; Caputo et al., 2015).

The geomorphological features of the studied area are sculptured into the alluvial plain formed by the digressions of a riverbed (Reno river), whose course was continuously changed by man in the past centuries with the aim to control its disastrous floods (Bondesan, 1990; 2001). The original river course was definitely abandoned in the 18th century, so the area where the San Carlo village lies is characterized by the old riverbed and earthen embankments degrading toward the alluvial plain (Fig. 2a,b). Due to the past river digressions, the sediments are characterized by a complex succession of alluvial deposits belonging to the depositional environments of river channel, river embankment, river rout and floodplain.
The local Emilia-Romagna stratigraphic sequence is divided into an upper and a lower sub-sequence. The upper sub-sequence is divided into 8 members, whose youngest two characterize the local stratigraphical succession of the studied area. The Holocene deposits are approximately 20 m thick and formed by sandy-silty deposits of river channel, river embankment, river rout and by clayey-silty deposits of floodplain with frequent lateral heterogeneities. The upper Pleistocene deposits follow beneath with an approximate thickness of 60 m, formed by a multilayering of fine sediments of marsh origin and coarser sediments of river overflowing. The complex soil layering does not allow an easy reconstruction of the lithological succession, characterized by frequent lateral and vertical variations (Romeo et al., 2015). Due to the complex soil layering, the hydrogeological features are also complex since the ground water table, oscillating within the first 10 m depth, is hosted in a multilayer aquifer. Although there is a correlation between the ground slope and the direction of drainage, their different gradients imply that the ground water depth is maximum below the old riverbed and progressively decreases toward the floodplain, thus implying different stress conditions in the subsoil. A monitoring period of two years, whose data are available thanks to the Emilia region technical offices, highlights that the ground water table oscillations are limited to less than 2 meters, with a shallower depth during the spring and the deepest one at the end of the summer season. The area is studied by extensive field investigations mainly consisting of penetrometer tests, integrated by boreholes and some dilatometer tests available in several reports and already collected by Papathanassiou et al. (2015). More in particular, the Liquefaction Potential Index (LPI) and Liquefaction Severity Number (LSN) indexes (Iwasaki et al., 1978; Toprak and Holzer, 2003) have been demonstrated to be suitable for detecting the tendency of different subsoil conditions to generate, or not, liquefaction surface manifestations (Giannakogiorgios et
al., 2015). As reported in Papathanassiou et al. (2015), both LSN and LPI allow the identification of local liquefaction-prone conditions at San Carlo when compared to the observed liquefaction effects induced by the 2012 Emilia earthquake (Fig.1a, Table 1). Along the here considered geological cross section (Fig.2b), the resulting LPI values vary from 11.2 up to 25.7 while the LSN values range from 8.2 up to 27.6. According to Papathanassiou et al. (2015), the so resulting correlation classes vary from 3 to 4, identifying a proneness to soil liquefaction from “likely” to “very likely”.

On the basis of the available investigations, a geological model of the investigated site is reconstructed (Romeo et al., 2015). Fig. 2c shows a NW-SE geological profile across the San Carlo village running parallel to the old river bed. As many investigations refer to indirect geotechnical tests (cone penetration test and dilatometer test), they have been interpreted with the perspective of highlighting the soil layers susceptible to liquefaction. In the case of penetrometer tests, the soil classification by Robertson (1986, 1990) is adopted, as it has been found to be most consistent with the liquefaction susceptibility. Having sampled the sands ejected during the liquefaction effects, their grain size distribution has been used to identify the Robertson’s classes indicating liquefaction susceptibility. More in particular, according to Robertson (1986; 1990), classes 7-8-9 are obtained for liquefiable soils, based on CPTU, and 5-6 based on SCPTU; classes 1-2-3-4-5-6-10 are obtained for not liquefiable soils, based on CPTU, and 1-2-3-4-7-8-9 based on SCPTU. The layers susceptible to liquefaction correspond to the grain size classes of sands, silty sands and sandy silt. Based on this classification, the layers belonging to the liquefiable classes are grouped by depth. Due to the stratigraphic succession of liquefiable soil layers and the resulting confining pressures, we can infer that only the first two surficial layers could have been involved in liquefaction phenomena. The geological model...
highlights that three different columns could represent the local typical layering: 1) a single-
layer of surficial liquefiable soils; 2) a double-layer of thin liquefiable soils; 3) a double-layer
with one thick and one thin liquefiable soils.

The local log-stratigraphy pointed out that within the first 15 meters below the ground, there are
one or two layers of liquefiable soils dating to Holocene and representing soils of river
embankment and river rout (Fig. 2c). Another layer of liquefiable soils dating back to
Pleistocene is present at depths greater than 20 m below the ground (Fig. 2c). Except for the first
surficial layer, the other layers are partially or fully confined by non-liquefiable soils (clays and
silty clays).

According to the aforementioned cross section, it can be derived that two sandy-silty layers
(hereafter named LS1 and LS2 respectively) are present in the San Carlo village area within the
first 20 m below the ground level. The first layer (LS1) is a superficial deposit with a thickness
varying from about 5 up to 10 m and corresponds partly to the recent alluvial deposits of the
Reno River and its tributaries, and partly to the old river banks. The thickness of the second,
deeper layer (LS2) is variable and ranges between 1 and 7 m, and this layer can be attributed to
more ancient alluvial deposits. More in particular, LS1 level is a continuous layer prone to
liquefaction whose thickness has a significant variation due to the topographic irregularities,
which can be mainly related to the paleo-morphology of the old river banks. Moreover, due to
its stratigraphic location, after the seismic shaking, this layer could dissipate the originated pore
water overpressures only from the topographic surface. LS2 level is a more discontinuous layer,
whose thickness varies within a few meter range, i.e. probably related to floods from ancient
river banks. This level shows a not continuous shape along the transversal sections and, where
present, it is separated from the LS1 level by a level not susceptible to liquefaction, having a
thickness varying up to about 6 m. Due to such a stratigraphic setting, the LS2 level can be regarded as a not drained one, i.e. co-seismic drainage of pore water due to overpressures is not possible. Below the LS2 level an about 10 m thick level not susceptible to liquefaction exists, ascribable to the Pleistocene-Holocene; this level rests above a 5 m thick sandy-gravel level (at about -5 m a.s.l., i.e. 20 m b.g.l.), ascribable to the Pleistocene deposits.

Based on the engineering-geological model obtained for the San Carlo village three reference soil columns (C1, C2 and C3 of Fig. 2c) were derived for the 1D numerical modeling, which are summarized below:

- column C1: sandy-silt LS1 and LS2 levels (prone to cyclic mobility and liquefaction) are interlayered with two silty-clayey levels (not susceptible to cyclic mobility and liquefaction) which rest above a sandy-gravel level at about 20 m b.g.l.. This column was obtained by attributing to the silty-clayey level between LS1 and LS2 the lowest assumable thickness (i.e. equal to 2.3 m).

- column C2: sandy-silt LS1 and LS2 levels (prone to cyclic mobility and liquefaction) are interlayered with two silty-clayey levels (not susceptible to cyclic mobility and liquefaction) which rest above a sandy-gravel level at about 20 m b.g.l.. This column was obtained by attributing to the silty-clayey level between LS1 and LS2 the highest assumable thickness (i.e. equal to 5.2 m).

- column C3: a unique sandy-silt LS1+LS2 level prone to liquefaction is considered with the highest assumable thickness (i.e. equal to 12 m).

The safety factor (SF) against liquefaction has been computed for each column by the code CLiq v1.7 (Geologismiki®) according to the method of Robertson & Wride (1998), based on the ratio of demand (cyclic stress ratio) and capacity (cyclic resistance ratio). Only the sandy layers
between 5 and 10 m deep resulted to be liquefiable (SF < 1) in agreement with the results obtained by Facciorusso et al. (2015).

**ONE-DIRECTIONAL THREE-COMPONENT WAVE PROPAGATION MODEL**

The subsoil is assumed as horizontally layered (as it can be confirmed by the vertical section shown in Fig. 2c) and is modeled as a 1D soil profile (Fig. 3), considering its very large lateral extension. The multilayered soil is assumed infinitely extended along the horizontal directions $x$ and $y$ and, consequently, no strain variation is considered in these directions. A three-component seismic wave propagates vertically in $z$-direction from the top of the underlying elastic bedrock to the free surface. The soil is assumed to be a continuous medium, with nonlinear constitutive behavior.

The soil profile is discretized, using a finite element scheme, into quadratic line elements having three translational degrees of freedom per node. The finite element model applied in the present research is completely described in Santisi d’Avila et al. (2012).

The subsoil layers are bounded at the bottom by a semi-infinite elastic medium, representing the seismic bedrock. The absorbing boundary condition proposed by Joyner & Chen (1975) is applied at the soil-bedrock interface to take into account the finite rigidity of the bedrock and allows energy to be radiated back into the underlying medium. The bedrock is characterized by the shear and pressure wave velocities in the medium and density. The 3C velocity time histories at the bedrock level are obtained by deconvolution of a seismic signal representative of outcropping bedrock. The soil motion at the soil-bedrock interface, i.e. at the first node of the mesh (Fig. 3), is composed of the incident and reflected waves and it is computed during the process. At this regard, the interested reader can refer to Santisi d’Avila et al. (2012) for more
details. As the considered horizontally layered subsoil is bounded at the top by the free surface, the stresses normal to the free surface are assumed to be null.

The finite element size in each soil layer is defined as the minimum between 1m and $\lambda/p$, where $\lambda = v_{si}/f$, $p = 10$ is the minimum number of nodes per wavelength to accurately represent the seismic signal, $v_{si}$ is the shear wave velocity in the $i$-th soil layer and $f = 15$Hz is the maximum frequency above which the spectral content of the input signal can be considered negligible. The number of finite elements per layer takes into account the expected reduction of the shear wave velocity $v_{s}$, during the dynamic process, that modifies the wavelength $\lambda = v_{s}/f$.

The implicit dynamic process is solved step-by-step by Newmark’s algorithm. The two parameters $\beta = 0.3025$ and $\gamma = 0.6$ guarantee an unconditional numerical stability of the time integration scheme (Hughes, 1987) and numerical damping, to reduce the not physical high frequency content numerically generated without having any significant effect on the meaningful, lower frequency response. According to Hughes (1987), the numerical damping is about 3% at 10Hz. Moreover, the nonlinearity of soil demands the linearization of the constitutive relationship within each time step. The discrete dynamic equilibrium equation does not require an iterative solving, within each time step, to correct the tangent stiffness matrix, if a small fixed time step $dt = 10^{-4}$s is selected. Gravity load is imposed as static initial condition in terms of strain and stress.

FEATURES OF THE 3D NONLINEAR HYSTERETIC MODEL

The 3D elasto-plastic model for soils used in the presented finite element scheme is inspired from that suggested by Iwan (1967) and applied by Joyner (1975) and Joyner and Chen (1975) in a finite difference formulation, in terms of total stresses.
The adopted model for TSA of soils satisfies the so-called Masing criteria (Kramer, 1996) that does not depend on the number of loading cycles. As a consequence, in a total stress analysis, the effects of the soil nonlinearity could be overestimated reducing the maximum strain values. According to Joyner (1975), the tangent constitutive matrix is deduced from the actual strain level and the strain and stress values at the previous time step. Then, the knowledge of this matrix allows calculating the stress increment. Consequently, the stress level depends on the strain increment and strain history but not on the strain rate. Therefore, this rheological model has no viscous damping. The energy dissipation process is purely hysteretic and does not depend on the frequency.

A correction of mechanical properties is applied according to Iai’s model (Iai et al., 1990a,b), for liquefiable soil layers. Iai’s rheological model for saturated soils allows attaining larger strains with proper accuracy describing the cyclic mobility process. This correction for liquefiable soils results in a reduction of hysteretic damping overestimation due to the Masing criteria and an increase of the expected maximum strains.

**Plasticity model**

Iwan’s model is a 3D elasto-plastic model with linear kinematic hardening, that allows taking into account the nonlinear hysteretic behavior of soils. Elastic parameters are the shear modulus

\[ G_0 = \rho v_s^2 \] (where \( \rho \) is the mass density and \( v_s \) the shear wave velocity in the medium) and the P-wave modulus

\[ M = \rho v_p^2 \] (where \( v_p \) is the pressure wave velocity in the medium). The Poisson’s ratio \( \nu \) is related with the compressional to shear wave velocity ratio as

\[ \left( \frac{v_p}{v_s} \right)^2 = 2(1-\nu)/(1-2\nu). \]

The 6-dimensional vector of total deviatoric strain rate \( \Delta e \) is written in terms of the elastic and
plastic deviatoric strain rates as \( \Delta e = \Delta e_E + \Delta e_p \). The elastic deviatoric strain vector is

\[
\Delta e_E = \frac{S}{2G_0}
\]  

(1)

where \( S \) is the 6-dimensional deviatoric stress vector.

The plasticity model uses von Mises yield surfaces that assume a pressure-independent behavior, namely, yielding is independent of the average pressure stress \( p \). This assumption is acceptable for soils in undrained conditions, as during a sudden seismic event. However, the Iai’s correction for effective stress analysis is pressure-dependent.

A family of \( n \) yield surfaces is represented by the yield functions

\[
F_i(S - \alpha_i) - \tau_i^2 = 0
\]  

(2)

where \( \alpha_i \) is the kinematic shift of the \( i \)th yield surface and \( F_i(S - \alpha_i) \) is the \( i \)th von Mises yield surface defined as

\[
F_i(S - \alpha_i) = \frac{1}{2}(S - \alpha_i)^T(S - \alpha_i)
\]  

(3)

The size of the yield surface \( \tau_i = \tau_i(e_i) \) is imposed by giving the value of the yield shear stress \( \tau_i \), as a function of shear strain \( e_i \), in the case of simple shear.

The kinematic hardening models are used to simulate the inelastic behavior of materials that are subjected to cyclic loading. The linear kinematic model approximates the hardening behavior with a constant rate of hardening, as expressed by the Prager hardening rule

\[
\Delta \alpha_i = C_i \Delta e_{p_i}
\]  

(4)

where \( C_i \) are the initial kinematic hardening moduli for each back stress \( \alpha_i \), defined as (Joyner, 1975)

\[
\frac{1}{C_i} = \frac{e_{i+1} - e_i}{\tau_{i+1} - \tau_i} = \frac{1}{2G} \sum_{j=1}^{i-1} \frac{1}{C_j}
\]  

(5)
The plasticity model assumes an associated plastic flow, which allows for isotropic yield. Therefore, as the material yields, the inelastic deformation rate is in the direction of the normal to the yield surface (the plastic deformation is volume invariant). The rate of plastic flow $\Delta e_p$ is defined by the following flow rule:

$$\Delta e_{pi} = L_i \Delta \lambda_{pi} \frac{\partial F_i}{\partial S}$$  \hspace{1cm} (6)

The coefficient $L_i$ is defined as

$$L_i = 0 \text{ for } F_i < \tau_i^2 \text{ or } \frac{\partial F_i}{\partial S} dS < 0$$

$$L_i = 1 \text{ for } F_i = \tau_i^2 \text{ and } \frac{\partial F_i}{\partial S} dS \geq 0$$  \hspace{1cm} (7)

and $\Delta \lambda_{pi}$ is the plastic strain rate, that is deduced, according to Fung (1965) as

$$\Delta \lambda_{pi} = \frac{1}{C_i} \frac{\partial F_i}{\partial \Delta S_{rs}} \Delta S_{rs}$$  \hspace{1cm} (8)

Consequently, the terms of the plastic deviatoric strain $\Delta e_{pi}$ in equation (6) become

$$\Delta e_{pi,hj} = \left[ \sum_{i=1}^{n} L_i \frac{1}{C_i} \frac{\partial S_{rs}}{\partial S_{rs}} \frac{\partial F_i}{\partial S_{ls}} \frac{\partial F_i}{\partial S_{lj}} \right] \Delta S_{rs}$$  \hspace{1cm} (9)

Writing the incremental constitutive relationship as

$$\Delta e = E_d^{-1} \Delta S$$  \hspace{1cm} (10)

and considering equations (1) and (9), the terms of the inverse deviatoric constitutive matrix $E_d^{-1}$ are deduced as
\[
\frac{1}{2G_0} + \sum_{i=1}^{n} \left( L_i \frac{\partial F_i}{\partial S_{zz}} \frac{\partial F_i}{\partial S_{ij}} \right) \tag{11}
\]

Coefficients in equation (11) are the derivatives of functions \( F_i \) in equation (3), that are

\[
\frac{\partial F_i}{\partial S_{jk}} = \frac{1}{2} \left( S_{jk} - \alpha_{ijk} \right) \tag{12}
\]

When the deviatoric constitutive matrix is known, it is possible to evaluate the deviatoric stress rate as \( \Delta S = E_d \Delta e \). The \((6 \times 6)\)-dimensional tangent constitutive matrix \( E \), that relates the total stresses and strains in the form \( \Delta \sigma = E \Delta e \), is obtained from \( E_d \) according to Santisi d’Avila et al. (2012).

According to Joyner (1975), to ensure that the stress remains on the yield surface, \( \Delta \alpha_i \) is not calculated using equation (4), but the following relationship:

\[
\alpha_{r,k+1} = S_{r,k+1} - \frac{\tau_i \left( S_{k+1} - \alpha_{r,k} \right)}{\sqrt{1/2 \left( S_{k+1} - \alpha_{r,k} \right)^T \left( S_{k+1} - \alpha_{r,k} \right)}} \tag{13}
\]

The failure curve \( \Delta \tau(\gamma) = G(\gamma) \Delta \gamma \), where \( G(\gamma) \) is the shear modulus decay curve versus shear strain \( \gamma \) is needed to characterize the soil behavior. The main feature of Iwan’s model is that the mechanical parameters to calibrate the rheological model are easily obtained from laboratory dynamic tests on soil samples.

The applied constitutive model does not depend on the selected initial loading curve. In the present study, normalized shear modulus decay curves are provided by laboratory tests, as resonant column (RC), and fitted by the function \( G(\gamma)/G_0 = 1/(1+|\gamma/\gamma_{r,0}|) \), where \( \gamma_{r,0} \) is a reference shear strain corresponding to an actual tangent shear modulus \( G(\gamma) \) equivalent to 50%
of the elastic shear modulus $G_0$. This model provides a hyperbolic stress-strain curve (Hardin and Drnevich, 1972), having asymptotic shear stress $\tau_0 = G_0 \gamma_{r0}$ in the case of simple shear. If no additional information is available, the normalized compressional modulus reduction curve $E/E_0$ is assumed equal to the shear modulus reduction curve $G/G_0$, under the commonly agreed hypothesis of constant Poisson’s ratio during the time history.

**Correction of mechanical parameters for soils in ESA**

Effective stresses follow Terzaghi’s law. Note that in the presented formulation the prime indicates effective stresses. The average effective stress is defined as $p' = (\sigma'_1 + \sigma'_2 + \sigma'_3)/3 = p - u$, where $p = (\sigma_1 + \sigma_2 + \sigma_3)/3$ is the average total stress, $\sigma_1$, $\sigma_2$ and $\sigma_3$ are the principal stresses and $u$ is the pore water pressure. The initial average effective stress is $p'_0$. The deviatoric stress is $\tau = (\sigma_{\text{max}} - \sigma_{\text{min}})/2 = \tau'$, where $\sigma_{\text{max}}$ and $\sigma_{\text{min}}$ are the maximum and minimum principal stresses, respectively.

The liquefaction front in the plane $(r, S)$ is represented in Fig. 4, where $S = p'/p'_0$ is the state variable, with $0 \leq S \leq 1$, and $r = \tau/p'_0$ is the deviatoric stress ratio. According to Iai et al. (1990a,b), the state variable $S$ relates the initial and the actual average effective stress and it is expressed as

$$S = \begin{cases} S_0 & r \leq r_3 \\ S_2 + \sqrt{(S_0 - S_2)^2 + \left[(r - r_3)/m_1\right]^2} & r > r_3 \end{cases}$$ (14)a,b

where $m_1 = \tan \alpha = \sin \phi'$ is the failure line slope (Fig. 4) and $\phi'$ is the shear friction angle. It can be remarked in Fig. 4 that $(r_2 - r_3)/(S_0 - S_2) = m_1$. Accordingly, the parameter $S_2$ is obtained as
\[ S_2 = S_0 - (r_2 - r_3)/m_1 \]  \hspace{1cm} (15)

In equation (15), it is \( r_2 = m_2 S_0 \), \( r_3 = m_3 S_0 \), \( m_3 = 0.67 m_2 \) and \( m_2 = \tan \alpha_p = \sin \phi'_p \) is the phase transition line slope (Fig. 4), where \( \phi'_p \) the phase transformation angle.

The initial value of liquefaction front parameter \( S_0 \) is determined by imposing the initial condition \( S = 1 \) in equation (14)b, according to (15). In dry and non-liquefiable layers, it is \( S = 1 \) during the seismic event.

Iai et al. (1990a,b) provide a relationship to correlate the liquefaction front parameter \( S_0 \) and the normalized shear work \( w \), as follows:

\[
S_0 = \begin{cases} 
(0.4 - S_1)\left(\frac{w_i}{w}\right)^{p_1} + S_1 & w \geq w_i \\
1 - 0.6\left(\frac{w}{w_i}\right)^{p_1} & w < w_i, \ S_1 = 1
\end{cases}
\]  \hspace{1cm} (16)a,b

Accordingly, it is

\[
w = w_i \left[ \frac{(S_0 - S_1)/(0.4 - S_1)}{S_0 \leq 0.4} \right]^{1/p_2} \]

\[
w = w_i \left[ \frac{(1 - S_0)/0.6}{S_0 > 0.4} \right]^{1/p_1}
\]  \hspace{1cm} (17)a,b

The normalized shear work is \( w = W/W_n \). The normalization factor is \( W_n = \left( p/m_1 \gamma_{r_0} \right)^2 \), where \( \gamma_{r_0} \) is the reference strain used in the hyperbolic formulation adopted for the backbone curve. The plastic shear work \( W \) is unknown at the initial conditions and it is estimated from \( w \) (equation (17)) and \( W_n \). The correlation between \( S_0 \) and \( w \), in equation (17), depends on four material parameters \( S_i, w_i, p_1 \) and \( p_2 \) that characterize the liquefaction properties of the cohesionless soil.

The main process starts with the computation of the actual plastic shear work. The increment of plastic shear work at each time step is
\[ dW_s = R \left( dW_{st} - c_1 dW_{se} \right) \geq 0 \]  \hspace{1cm} (18)a,b

where, according to Towhata and Ishihara (1985), the shear stress work \( dW_{st} \) is evaluated as the difference between the total work \( dW = \sigma'_{ij} d\varepsilon_{ij} \) and the consolidation work \( dW_c = p'd\varepsilon_v \), where \( d\varepsilon_v \) is the volumetric strain. There exists a threshold limit in the amplitude of cyclic shear strain or shear stress. There is no pore water pressure build-up for cyclic strain or stress below this threshold level. The shear work consumed by the threshold limit is subtracted from the total shear work. It is closely related with the elastic shear work \( dW_{se} = \left| \tau d\left( \tau/G \right) \right| \). The parameter \( c_1 \approx 1 \) is introduced to correct the elastic shear work \( dW_{se} \) for the purpose of obtaining the shear work consumed by the threshold limit. \( R \) is a correction factor for \( dW_s \) in the case of dilatancy, that means \( \tau > p_0 m_2 \), and \( r/S > m_3 \). It is defined as

\[
R = \frac{(m_i - r/0.4)}{(m_i - m_3)} \quad S_0 \leq 0.4 \\
R = \frac{(m_i - r/S)}{(m_i - m_3)} \quad S_0 > 0.4 
\]  \hspace{1cm} (19)a,b

When the actual plastic shear work \( W_s \) is known, the normalized shear work \( w \) is evaluated, the liquefaction front parameter \( S_0 \) is deduced from equation (16) and the state variable \( S \) is obtained by equation (14). According to the definition of \( S \), the actual average effective stress \( p' = S p_0' \) and the increment of water pressure \( \Delta u = p_0' - p' > 0 \) are obtained during the time history. The actual effective stress \( \sigma' = \sigma - u \) is deduced from the total stress and water pressure \( u = p - p' \). Finally, the updated deviatoric stress \( \tau_a \) and the reference shear strain \( \gamma_{ra} \) are estimated as

\[
\tau_a = \tau_0 S \\
\tau_a = \tau_0 S + \Delta \tau = \tau_0 S + \tau_0 \left( 1 - m_i/m_2 \right) (0.4 - S_0) \\
\gamma_{ra} = \gamma_{r0} \\
\gamma_{ra} = \gamma_{r0} / (S_0/0.4) \\
\]  \hspace{1cm} (20)a,b
The corrected shear modulus is

\[ G_u = \frac{\tau_a}{\gamma ra} \]  

(21)

Consequently, the normalized shear modulus decay curve is updated as

\[ G(\gamma)/G_u = \frac{1}{1 + |\gamma/\gamma ra|}. \]

Characterization of soil parameters in ESA

Seven parameters have to be fixed to calibrate Iai’s correction of shear modulus of soils for ESA. They are the shear friction angle \( \phi' \), the phase transformation angle \( \phi'_p \), the parameter \( c_i \) that corrects the elastic shear work and the four parameters \( S_1, w_1, p_1 \) and \( p_2 \) that influence the relationship between the liquefaction front parameter \( S_0 \) and the normalized shear work \( w \).

The shear friction angle \( \phi' \) and the phase transformation angle \( \phi'_p \) are obtained from static Consolidated Undrained (CU) triaxial tests, using the \( (\tau, p') \) curve for three different confining pressure levels. The slope of the line connecting the rupture points, for the three different confining pressure levels, is the trigonometric tangent of angle \( \alpha \). The slope of the line connecting the inflection points of the three curves is the trigonometric tangent of angle \( \alpha_p \). The shear friction angle \( \phi' \) and the phase transformation angle \( \phi'_p \) are obtained considering the equivalences \( \tan \alpha = \sin \phi' \) and \( \tan \alpha_p = \sin \phi'_p \), respectively.

According to Iai et al., 1990a,b, parameters \( S_1, w_1, p_1 \) and \( p_2 \) are deduced from CTX tests.

Three curves have to be reproduced: the cyclic deviatoric stress, the deviatoric strain amplitude and the normalized excess pore water pressure \( \Delta u/p'_0 \) with respect to the number of cyclic loading \( N \), where \( \Delta u = p'_0 - p' \) is the excess pore water pressure.
MODEL OF SELECTED SOIL PROFILES

The definition of the three analyzed soil profiles, named here C1, C2 and C3, is derived from the stratigraphy in the San Carlo area, at about 17 km from the epicenter of the 2012 Emilia earthquake. The selected stratigraphy for the three columns C1, C2 and C3, and the mechanical parameters identified for each layer are presented in Tables 2, 3 and 4, respectively. The front liquefaction parameters are also reported for cohesionless soil layers, subjected to possible cyclic mobility effects and liquefaction phenomena. Liquefiable soil layers are the 1st, 3rd and 5th in soil columns C1 and C2, and the 1st and 3rd in C3.

An in-situ test using seismic dilatometer has been used to obtain profiles with depth of the shear wave velocity $v_s$, total (wet) density $\rho$ and at-rest lateral earth pressure coefficient $K_0$. The initial elastic shear modulus $G_0$ is evaluated as $G_0 = \rho v_s^2$. The elastic P-wave modulus is evaluated as $M_0 = \rho v_p^2$, where the compressional wave velocity $v_p$ is deduced from the relationship $v_p = v_s \sqrt{2(1-\nu)(1-2\nu)}$, where the Poisson’s ratio is obtained as $\nu = K_0/(1+K_0)$.

Laboratory tests are used to obtain the mechanical features of each soil layer, useful to define the initial conditions in the numerical simulation. A RC test gives the normalized shear modulus reduction curve $G(\gamma)/G_0$ that is fitted using a curve corresponding to a hyperbolic stress-strain curve. The reference shear strain $\gamma_{r0}$ is deduced as the strain corresponding to $G = 0.5 G_0$. The shear modulus reduction curves obtained by laboratory data are given in Table 5, for the soil samples used to characterize the analyzed soil columns. The normalized curves, fitted by the hyperbolic model are shown in Fig. 5.

The variation of the elastic shear modulus with depth is taken into account for the liquefiable
soil layers. It is corrected according to 
\[ G_0(z) = G_0(z_m) \frac{p'_0(z)}{p'_0(z_m)}, \]
where \( p'_0 \) is the average pressure, \( z_m \) is the depth at the middle of the layer and 
\[ G_0(z_m) = \rho v_s^2 \]
is calculated using the values of density and shear velocity reported in Tables 2-4 for each soil layer. As consequence, the shear strength \( \tau_0 = G_0 \gamma_r \) is modified. In non-liquefiable layers, considering that their thickness is limited, the shear modulus is assumed constant with depth.

A CU triaxial test provides the shear friction angle \( \phi' \) and the phase transformation angle \( \phi'_p \) using a curve \((\tau, p')\) for three different confining pressure levels. The slope of the line connecting the rupture points, for the three different confining pressure levels, is the trigonometric tangent of the shear friction angle \( \phi' \). The slope of the line connecting the inflection points of the three curves is the trigonometric tangent of the phase transformation angle \( \phi'_p \).

The CTX test gives the liquefaction front parameters \( c_1, S_1, w_1, p_1 \) and \( p_2 \) as explained in the next subsection.

The average effective stress in geostatic conditions \( p'_0 = (\sigma'_v + 2 \sigma'_h)/3 \) is evaluated considering the vertical effective stress \( \sigma'_v(z) = \rho g z - u_0(z) \), variable with depth \( z \) (\( g \) is the gravitational acceleration) and dependent on the initial pore water pressure \( u_0(z) \), and the horizontal effective stresses estimated as \( \sigma'_h(z) = K_0 \sigma'_v(z) \). At the surface, where the vertical stress attains zero but the confinement, even if reduced, is not annulled, the horizontal effective stresses are corrected and assumed equal to \( \sigma'_h = K_0 \sigma'_v(z_p) \) in the first \( z_p \) meters (in this study, it is assumed \( z_p = 5 \text{ m} \)).
Fitting of cyclic consolidated undrained triaxial test curves

The parameters $S_1$, $w_1$, $p_1$ and $p_2$ are deduced from CTX tests. The cyclic deviatoric stress (total axial stress minus confining pressure) produced during the test is used to estimate the total axial stress that is adopted as input in a numerical simulation, known the cell pressure $p_0 = 300\text{kN/m}^2$ and the back pressure $u_0 = 200\text{kN/m}^2$ during the test. The axial deviatoric strain amplitude and the normalized excess pore water pressure $\Delta u/p_0'$ ($\Delta u = p_0' - p'$ is the excess pore water pressure), with respect to the number of cyclic loading $N$, are obtained numerically and compared to the two curves produced during the test.

In order to obtain numerically the curves that best reproduce the experimental ones, the parameters $w_1$ and $p_1$ are determined by a trial-and-error procedure, to obtain a normalized excess pore water pressure curve that best reproduce the experimental curve in the portion of the curve for $\Delta u/p_0' < 0.6$. The parameter $w_1$ is not greatly influenced by the variation of $p_1$, so it is determined at first for a given value of $p_1$. The appropriate value of $p_1$ is researched in the interval $[0.4 - 0.7]$, according to Iai et al. (1990b). The greater $w_1$ and $p_1$ are, the slower the pore water pressure rises. The envelope of strain amplitude is also fitted, observing that the greater $w_1$ is, the more it reduces the envelope of strain amplitude.

The parameter $p_2$ is researched in the interval $[0.6 - 1.5]$ (Iai et al. 1990b). It is determined as well by a trial-and-error process, to obtain a normalized excess pore water pressure curve that best fit the experimental curve in the portion of the curve for $\Delta u/p_0' > 0.6$. Since the curve is not greatly influenced by the variation of $p_2$, the envelope of strain amplitude is also fitted. The greater $p_2$ is, the more it increases the envelope of strain amplitude.
According to Iai et al. (1990b), the parameter $S_i \geq 0.005$ is introduced so that $S_i$ will never be zero. It takes small positive values, determined by the trial-and-error procedure to obtain the best fit of the experimental normalized excess pore water pressure curve. The analyzed tests appear not sensitive to a variation of $S_i$. The first trial is maintained for all the layers: $S_i = 0.005$. The parameter $c_i$ is imposed equal to one when $w_i$, $p_i$ and $p_2$ are determined and, if laboratory data are not well represented in the elastic range, $c_i$ can be modified using a trial-and-error procedure. The mechanical parameters measured by laboratory tests (RC, CU) are listed in Table 6 for the analyzed soil samples S11-C1 (associated to the liquefiable layer LS1 in Tables 2-4) and S11-C3 (associated to LS2). The selected liquefaction parameters for S11-C1 and S11-C3 are the average between the values obtained by calibration for the two available tests (listed in Table 6) and these selected values are assumed constant with depth within each liquefiable soil layer. The fitting of CTX test curves is shown in Fig. 6 for the S11-C1-2 soil sample (see Table 6) and in Fig. 7 for the S11-C1-3 soil sample. Figs 6 and 7 show the measured cyclic axial deviatoric stress, applied as input, the measured and numerical axial deviatoric strain amplitude and normalized excess pore water pressure with respect to the number of cyclic loading. The curves $(\tau, p')$ and $(r, S)$ are evaluated numerically.

**Input seismic motion**

Since in May 2012 the stations of the fixed Italian National Accelerometric Network (RAN) were not present in the municipal area of San Carlo, the mainshock was not recorded in this site. Even though the temporary array, installed the day after the mainshock by the Nacional Civil Protection, recorded 12 seismic events with $M_w$ higher than 4, no further liquefaction effects have been observed after the mainshock of 20 May 2012. Therefore, as reported by Romeo et al.
2015), the seismic input at the San Carlo site was derived by: 1) evaluating a spectrum of scenario for the site of San Carlo; 2) selecting the record of the 20 May 2012 mainshock at Mirandola as reference input and deconvolving it to bedrock and outcrop; 3) attenuating the record of Mirandola at the outcropping bedrock of San Carlo using the ground motion prediction equation proposed by Sabetta and Pugliese (1987).

The three-components of the reference outcropping motion are halved and applied as incident wavefield at the base of the analyzed soil columns. The reference incident motion applied at the soil-bedrock interface is shown in Fig. 8, in terms of acceleration. The peak acceleration is 2.54 m/s² in North-South direction (named \( x \) in the model), 1.51 m/s² in East-West direction (named \( y \)) and 0.33 m/s² in Up-Down direction (named \( z \)).

All input and output signals are filtered using a 4-pole Butterworth bandpass filter in the frequency range 0.1–15 Hz.

EEFECT OF WATER TABLE DEPTH VARIATION

First, the seismic response of the analyzed soil profiles to the 20 May 2012 Mw 5.9 Emilia earthquake is estimated in terms of total stresses, to assess the extent of error in this study case where liquefaction phenomena are expected. Then, the analysis is developed in terms of effective stresses.

The assumed water table depth is \( z_w = 5.8 \) m for C1 column, \( z_w = 5.2 \) m for C2 column and \( z_w = 2.7 \) m for C3 column, according to the available technical reports. Moreover, a water table variation of ±1 m is considered in this research and its influence on the soil column response to the seismic loading is analyzed.
Profiles with depth of the peak acceleration and velocity, shear strain and stress for the soil profile C1 are represented in Fig. 9, in the cases of TSA and ESA with variable water table position. Strains are increased in liquefiable layers, compared with the TSA assumption. The shape of loops in liquefiable soil layers is influenced by the reduction of shear modulus during the cyclic mobility (Fig. 10 bottom). Profiles with depth and hysteresis loops for soil columns C2 and C3 are presented in Figs 11-12 and 13-14, respectively.

In liquefiable soil layers, the shear modulus is reduced during the cyclic mobility and the reference shear strain $\gamma_{r0}$ is numerically corrected when the liquefaction front parameter $S_0$ is lower than 0.4, according to Equation (20). The minimum values attained by the shear modulus during the process, at each depth, and the maximum reference shear strain are shown in Fig. 15, for the C3 soil profile.

Observing Figs 9-14, the considered variation of the water table position equal to ±1m is not influential in the seismic response of the analyzed soil profiles. In the analyzed case study, a total stress analysis, for a stratigraphy where there are liquefiable soil layers, totally modifies the seismic response, neglecting the reduction of soil stiffness and the increase of ground motion. According to Figs 9, 11, 13, 16 and 17 and Table 7, the TSA assumption underestimates the peak values of motion.

According to Figs 9, 11 and 13, the maximum shear strain level, estimated using the analysis in terms of effective stress, is 6.8%, 5.6% and 6.2% for C1, C2 and C3 soil profiles, respectively. Maximum acceleration profiles with depth are obtained during the process using unfiltered acceleration. Figs 16-17 show filtered horizontal acceleration at the ground surface. The variability in the peak ground acceleration with the water table position is negligible (see Table 7).
The expected level of horizontal peak ground acceleration, corresponding to about 0.32g according to the automated shakemaps (INGV, see Data and Resources), is numerically obtained for the C3 soil profile (Table 7).

Fig. 18a shows the profiles with depth of maximum excess pore water pressure, normalized with respect to the initial average effective pressure, for the three analyzed soil columns. The maximum excess pore water pressure attains 84%, 93% and 98% of the initial average effective pressure, respectively. The time history at the depth where the excess pore water pressure attains the maximum value is shown in Fig. 18b.

CONCLUSIONS

A one-directional propagation model of a three-component seismic wave (1D-3C approach), in a finite element scheme, is used to investigate the seismic response and stress-strain induced effects of three soil profiles derived from the stratigraphy of San Carlo village (Emilia Romagna, Italy). A representative record for the 20 May 2012 Mw 5.9 Emilia earthquake is applied as input motion at the soil-bedrock interface.

During the 20 May 2012 Mw 5.9 Emilia earthquake, liquefaction phenomena have been observed. Consequently, an analysis in terms of total stresses is not suitable. A constitutive behavior based on soil plasticity with hardening (Iwan’s model) is used, where the nonlinearity is described by the normalized shear modulus reduction curve. The shear modulus is corrected during the process, depending on the actual average effective stress (Iai’s model) to consider the cyclic mobility and dilatancy of sands.

The seismic response of the analyzed soil profiles is discussed in terms of profiles with depth of maximum shear strain and stress, peak of the horizontal motion and maximum excess pore water pressure.
pressure. Hysteresis loops in liquefiable soils and the time histories of the ground motion and the excess pore water pressure are obtained for the different hypothesis of the water table depth.

The impact of ESA soil modeling in the numerical seismic response of a soil profile, compared with a total stress analysis, is observed for the analyzed case study and shows that the TSA assumption underestimates the peak values of the ground motion.

The influence on the numerical seismic response and the stress-strain effects of the water table depth variability is investigated and it appears negligible in the analyzed case study.

The attained maximum shear strain level, estimated using nonlinear analysis in terms of effective stress, attains 6%, confirming the observed liquefaction effect. The expected level of horizontal peak ground acceleration, deduced from the automated shakemaps, is numerically reproduced.

The maximum excess pore water pressure, obtained numerically, attains 98% of the initial average effective pressure in C3 soil profile.

The discrepancy between geotechnical parameters, obtained by different in-situ and laboratory tests, forces to some modeling choices. Further experimental research should be necessary to guarantee the interdependence of geotechnical data issued by tests for different soil samples in the same area and to regulate the procedure allowing the transposition of measures in the modeled soil profile.

The numerical approach applied here can be regarded as a useful tool for identifying subsoil portion prone to soil liquefaction during earthquakes. This allows the outline of unstable zones for expected nonlinear effects in the framework of Seismic Microzonazion Studies. In this regard, high resolution engineering-geological modeling of the subsoil makes it possible to extend the local numerical results (i.e. output from a single soil column) to adjacent areas, based on the similarity of soil layering and hydrogeological conditions. Such an approach exemplifies
the efficiency of a multidisciplinary approach which merges geological, physical and engineer
features to quantify complex effects involving a multiphase rheological system.

DATA AND RESOURCES

Seismograms and soil stratigraphic setting used in this study were obtained in the framework of
the Project S_2-2012 by INGV-DPC 2012-2013 – UR4 titled: "Validation of Seismic Hazard
through observed data; Constraining OBservations into Seismic hazard (COBAS)" (scientific
responsible: Laura Peruzza; UR4 co-ordinator R.W. Romeo). The laboratory tests used for this
study were performed on commission by the C.G.G. Testing S.r.l. laboratory of Bologna (Italy).
The Italian catalogue of earthquake-induced ground effects (CEDIT) is available online at the
Shakemaps produced by the Italian National Institute of Geophysics and Volcanology (INGV)
are available online at the URL http://shakemap.rm.ingv.it/shake/index.html (last accessed
November 2017).

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REFERENCES

Bonilla, L.F., R.J. Archuleta and D. Lavallée (2005). Hysteretic and dilatant behavior of
cohesionless soils and their effects on nonlinear site response: field data observations and


Facciorusso, J., C. Madiai and G. Vannucchi (2014). Soil liquefaction analyses in a test-area


Region (Italy), *Ingegneria sismica*, vol. 2–3, Pàtron Editore, Bologna.


LIST OF FIGURE CAPTIONS

Figure 1. Liquefaction effects observed at San Carlo village after the 20 May 2012 Mw 5.9 earthquake: a, b) evidences of clay spread out on free field; c, d) evidence of clays spread out from building foundations causing damage to structures.

Figure 2. (a) Google Earth satellite view of the San Carlo village with trace of geological cross section and location of modelled soil columns. (b) geological map of the San Carlo village: 1) deposits of river channel; 2) deposits of river banks; 3) deposits of alluvial plain; 4) liquefaction sand boil; 5) liquefaction trench; 6) borehole with samples; 7) borehole without samples; 8) Seismic DilatoMeter Test (SDMT); 9) Cone Penetration Test with piezocone (CPTU); 10) Seismic Cone Penetrometric Test with piezocone (SCPTU); 11) Seismic Cone Penetrometric Test (SCPT). (c) Geological cross section: 1) Holocene liquefiable deposits; 2) Holocene not liquefiable deposits; 3) Pleistocene liquefiable deposits; 4) Pleistocene non liquefiable deposits; 5) not classified Pleistocene deposits; 6) water table; 7) liquefaction sand boil; 8) liquefaction trench; 9) borehole; 10) CPTU or SCPT test; 11) modelled soil columns.

Figure 3. Spatial discretization of a horizontally layered soil. The seismic loading applied at the bedrock level is a deconvolved outcropping signal in terms of velocity.

Figure 4. Liquefaction front $r(S)$, where $r$ is the deviatoric stress ratio and $S$ is the state variable.

Figure 5. Normalized shear modulus reduction curves obtained using RC test results (markers) and fitted using the hyperbolic model (solid line), for soil samples S2-C2 (left) having reference shear strain $\gamma_{r0} = 0.48\%$, S3-C3 (middle) having $\gamma_{r0} = 0.39\%$ and S10-C3 (right) having $\gamma_{r0} = 0.49\%$.

Figure 6. Fitting of cyclic Consolidated Undrained triaxial test curves to calibrate liquefaction
parameters, for S11-C1-2 soil sample in LS1 liquefiable soil layer.

**Figure 7.** Fitting of cyclic Consolidated Undrained triaxial test curves to calibrate liquefaction parameters, for S11-C3-3 soil sample in LS2 liquefiable soil layer.

**Figure 8.** Three components of the incident motion applied at the soil-bedrock interface in terms of acceleration. The peak acceleration is $2.54 \, \text{m/s}^2$ in $x$-direction, $1.51 \, \text{m/s}^2$ in $y$-direction and $0.33 \, \text{m/s}^2$ in $z$-direction.

**Figure 9.** Profiles with depth of maximum shear strain and stress, horizontal velocity and acceleration, during the seismic event, for different water table depth and for TSA conditions in the C1 soil column. The horizontal dashed lines indicate the depth where the shear stress-strain loops are analyzed.

**Figure 10.** Hysteresis loops in C1 soil profile for different water table depth and for TSA conditions: (top) at 21.5 m in a non liquefiable soil layer; (bottom) at 13 m in LS2 liquefiable soil layer.

**Figure 11.** Profiles with depth of maximum shear strain and stress, horizontal velocity and acceleration, during the seismic event, for different water table depth and for TSA conditions in the C2 soil column. The horizontal dashed lines indicate the depth where the shear stress-strain loops are analyzed.

**Figure 12.** Hysteresis loops in C2 soil profile for different water table depth and for TSA conditions: at 9 m (top) and at 12.5 m (bottom) in LS2 liquefiable soil layer.

**Figure 13.** Profiles with depth of maximum shear strain and stress, horizontal velocity and acceleration, during the seismic event, for different water table depth and for TSA conditions in the C3 soil column. The horizontal dashed lines indicate the depth where the shear stress-strain loops are analyzed.
Figure 14. Hysteresis loops in C3 soil profile for different water table depth and for TSA conditions: (top) at 20.5 m in a non liquefiable soil layer; (bottom) at 12 m in LS1+LS2 liquefiable soil layer.

Figure 15. Profile with depth of minimum shear modulus (left), during the seismic event, with zoom in the first soil layers (middle) and profile with depth of the maximum reference shear strain, for different water table depth and for TSA conditions, in the C3 soil profile. The horizontal dashed lines indicate the depth where the shear stress-strain loops are analyzed.

Figure 16. Acceleration time history at the ground surface for C1 (left), C2 (middle) and C3 (right) soil profiles, in x-direction, for different water table depth and for TSA conditions.

Figure 17. Acceleration time history at the ground surface for C1 (left), C2 (middle) and C3 (right) soil profiles, in y-direction, for different water table depth and for TSA conditions.

Figure 18. Excess pore water pressure, normalized with respect to the average effective pressure, for different water table depth, in C1 (left), C2 (middle) and C3 (right) soil columns: (top) Profiles with depth of maximum value during the seismic event. The horizontal dashed lines indicate the depth where the time histories are analyzed. (bottom) Time history: at 21.5 m (LS1) in C1 soil profile, at 20.5 m (LS2) in C2 soil profile and at 4.5 m (LS1+LS2) in C3 soil profile, for $z_w = 3.7$ m, at 3.5 m for $z_w = 2.7$ m and $z_w = 1.7$ m.
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Table 1. Values of LPI and LSN referred to the SCPTU and CPTU tests located in Fig. 2a and related correlation classes for proneness to soil liquefaction according to Papathanassiou et al., 2015: class-I: “almost no”, class-II: “few”, class-III: “likely” and class-IV: “very likely”.

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<th>In-situ test position</th>
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<th>LSN</th>
<th>Correlation class</th>
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Table 2. Stratigraphy and geotechnical parameters of soil profile C1. The liquefiable layers are referred as LS.

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<th>Layer</th>
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<th>Depth (m)</th>
<th>Density (kg/m³)</th>
<th>S-wave velocity (m/s)</th>
<th>P-wave velocity (m/s)</th>
<th>Earth press. coeff.</th>
<th>Reference strain (%)</th>
<th>Soil sample</th>
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<td>1800</td>
<td>180</td>
<td>465</td>
<td>0.7</td>
<td>0.48</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>sandy silt (LS2)</td>
<td>14.0</td>
<td>1850</td>
<td>180</td>
<td>402</td>
<td>0.6</td>
<td>0.39</td>
<td>S11-C3</td>
</tr>
<tr>
<td>4</td>
<td>silty clay</td>
<td>21.5</td>
<td>1800</td>
<td>200</td>
<td>632</td>
<td>0.8</td>
<td>0.48</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>sandy gravel</td>
<td>27.5</td>
<td>1850</td>
<td>275</td>
<td>710</td>
<td>0.7</td>
<td>0.39</td>
<td>S11-C3</td>
</tr>
<tr>
<td>6</td>
<td>silty clay</td>
<td>33.0</td>
<td>1900</td>
<td>280</td>
<td>723</td>
<td>0.7</td>
<td>0.49</td>
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</tr>
<tr>
<td>7</td>
<td>sandy gravel</td>
<td>73.0</td>
<td>1975</td>
<td>385</td>
<td>994</td>
<td>0.7</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>silty clay</td>
<td>113</td>
<td>2125</td>
<td>595</td>
<td>1536</td>
<td>0.7</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td></td>
<td>bedrock</td>
<td>&gt;113</td>
<td>2200</td>
<td>700</td>
<td>1807</td>
<td>0.7</td>
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</tr>
</tbody>
</table>
Table 3. Stratigraphy and geotechnical parameters of soil profile C2. The liquefiable layers are referred as LS.

<table>
<thead>
<tr>
<th>layer type</th>
<th>depth (m)</th>
<th>density (kg/m³)</th>
<th>S-wave velocity (m/s)</th>
<th>P-wave velocity (m/s)</th>
<th>earth press. coeff.</th>
<th>reference strain (%)</th>
<th>soil sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 sandy silt (LS1)</td>
<td>3.3</td>
<td>1700</td>
<td>200</td>
<td>894</td>
<td>0.9</td>
<td>0.39</td>
<td>S11-C1</td>
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<tr>
<td>2 silty clay</td>
<td>8.5</td>
<td>1750</td>
<td>190</td>
<td>601</td>
<td>0.8</td>
<td>0.48</td>
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</tr>
<tr>
<td>3 sandy silt (LS2)</td>
<td>12.5</td>
<td>1850</td>
<td>200</td>
<td>516</td>
<td>0.7</td>
<td>0.39</td>
<td>S11-C3</td>
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<tr>
<td>4 silty clay</td>
<td>20.5</td>
<td>1850</td>
<td>210</td>
<td>939</td>
<td>0.9</td>
<td>0.48</td>
<td></td>
</tr>
<tr>
<td>5 sandy gravel</td>
<td>26.0</td>
<td>1850</td>
<td>275</td>
<td>710</td>
<td>0.7</td>
<td>0.39</td>
<td>S11-C3</td>
</tr>
<tr>
<td>6 silty clay</td>
<td>33.0</td>
<td>1900</td>
<td>280</td>
<td>723</td>
<td>0.7</td>
<td>0.49</td>
<td></td>
</tr>
<tr>
<td>7 sandy gravel</td>
<td>73.0</td>
<td>1975</td>
<td>385</td>
<td>994</td>
<td>0.7</td>
<td>100</td>
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</tr>
<tr>
<td>8 silty clay</td>
<td>113</td>
<td>2125</td>
<td>595</td>
<td>1536</td>
<td>0.7</td>
<td>100</td>
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<tr>
<td>bedrock</td>
<td>&gt; 113</td>
<td>2200</td>
<td>700</td>
<td>1807</td>
<td>0.7</td>
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</table>

Table 4. Stratigraphy and geotechnical parameters of soil profile C3. The liquefiable layers are referred as LS.

<table>
<thead>
<tr>
<th>layer type</th>
<th>depth (m)</th>
<th>density (kg/m³)</th>
<th>S-wave velocity (m/s)</th>
<th>P-wave velocity (m/s)</th>
<th>earth press. coeff.</th>
<th>reference strain (%)</th>
<th>soil sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 sandy silt (LS1+LS2)</td>
<td>12</td>
<td>1800</td>
<td>200</td>
<td>516</td>
<td>0.7</td>
<td>0.39</td>
<td>S11-C3</td>
</tr>
<tr>
<td>2 silty clay</td>
<td>21.5</td>
<td>1850</td>
<td>190</td>
<td>491</td>
<td>0.7</td>
<td>0.48</td>
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</tr>
<tr>
<td>3 sandy gravel</td>
<td>25.0</td>
<td>1900</td>
<td>240</td>
<td>759</td>
<td>0.8</td>
<td>0.39</td>
<td>S11-C3</td>
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<td>4 silty clay</td>
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<td>1900</td>
<td>280</td>
<td>723</td>
<td>0.7</td>
<td>0.49</td>
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</tr>
<tr>
<td>5 sandy gravel</td>
<td>71.5</td>
<td>1975</td>
<td>385</td>
<td>994</td>
<td>0.7</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>6 silty clay</td>
<td>111.5</td>
<td>2125</td>
<td>595</td>
<td>1536</td>
<td>0.7</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Bedrock</td>
<td>&gt; 111.5</td>
<td>2200</td>
<td>700</td>
<td>1807</td>
<td>0.7</td>
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</table>
Table 5. Normalized shear modulus reduction curves $G/G_0$ obtained using RC tests.

<table>
<thead>
<tr>
<th>Soil sample</th>
<th>S2-C2</th>
<th>S3-C3</th>
<th>S10-C3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil type</td>
<td>sandy silt/gravel</td>
<td>silty clay</td>
<td>deep silty clay</td>
</tr>
<tr>
<td>Shear ref. strain</td>
<td>$\gamma_{r0} = 0.39%$</td>
<td>$\gamma_{r0} = 0.48%$</td>
<td>$\gamma_{r0} = 0.49%$</td>
</tr>
<tr>
<td>shear strain</td>
<td>%</td>
<td>G/G₀</td>
<td>shear strain</td>
</tr>
<tr>
<td>0.00015</td>
<td>1.000</td>
<td>0.00010</td>
<td>1.000</td>
</tr>
<tr>
<td>0.00030</td>
<td>1.000</td>
<td>0.00020</td>
<td>1.000</td>
</tr>
<tr>
<td>0.00064</td>
<td>1.000</td>
<td>0.00033</td>
<td>1.000</td>
</tr>
<tr>
<td>0.00097</td>
<td>0.995</td>
<td>0.00061</td>
<td>0.996</td>
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<tr>
<td>0.00161</td>
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<tr>
<td>0.00245</td>
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<td>0.00175</td>
<td>0.961</td>
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<td>0.926</td>
<td>0.00328</td>
<td>0.925</td>
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<td>0.00583</td>
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<td>0.00583</td>
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<td>0.01159</td>
<td>0.810</td>
<td>0.00905</td>
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<tr>
<td>0.02178</td>
<td>0.692</td>
<td>0.01688</td>
<td>0.719</td>
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<tr>
<td>0.04659</td>
<td>0.509</td>
<td>0.03310</td>
<td>0.529</td>
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<tr>
<td>0.07419</td>
<td>0.400</td>
<td>0.07133</td>
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<td>0.20062</td>
<td>0.197</td>
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**Table 6.** Liquefaction parameters used to calibrate CTX test curves

<table>
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<tr>
<th></th>
<th>Soil sample</th>
<th>S11-C1-1</th>
<th>S11-C1-2</th>
<th>S11-C3-2</th>
<th>S11-C3-3</th>
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</thead>
<tbody>
<tr>
<td>Density $\rho$ kg/m$^3$</td>
<td></td>
<td>1950</td>
<td>1950</td>
<td>2050</td>
<td>2050</td>
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<tr>
<td>S-wave velocity $v_s$ m/s</td>
<td></td>
<td>175</td>
<td>175</td>
<td>152</td>
<td>152</td>
</tr>
<tr>
<td>P-wave velocity $v_p$ m/s</td>
<td></td>
<td>2480</td>
<td>2480</td>
<td>2149</td>
<td>2149</td>
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<tr>
<td>Reference strain $\gamma_r$ %</td>
<td></td>
<td>0.11</td>
<td>0.11</td>
<td>0.15</td>
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<tr>
<td>Friction angle $\alpha$ °</td>
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<td>29</td>
<td>29</td>
<td>32</td>
<td>32</td>
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<tr>
<td>Dilatation angle $\phi_p$ °</td>
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<td>25</td>
<td>26</td>
<td>26</td>
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<tr>
<td>Cell pressure $p_0$ kN/m$^2$</td>
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<td>300</td>
<td>300</td>
<td>300</td>
<td>300</td>
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<tr>
<td>Back pressure $u_0$ kN/m$^2$</td>
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<td>200</td>
<td>200</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>$c_1$</td>
<td>5.0</td>
<td>4.9</td>
<td>5.2</td>
<td>5.0</td>
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<tr>
<td>$S_1$</td>
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<td>0.005</td>
<td>0.005</td>
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<td>$p_1$</td>
<td>0.4</td>
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<tr>
<td>$p_2$</td>
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<tr>
<td>$w_1$</td>
<td>5.9</td>
<td>6.2</td>
<td>6.5</td>
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</table>

**Table 7.** Peak ground acceleration in x- and y-direction for the different water table depth $z_w$

<table>
<thead>
<tr>
<th>Soil profile</th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$z_w$</td>
<td>$a_x$</td>
<td>$a_y$</td>
</tr>
<tr>
<td></td>
<td>m</td>
<td>m/s$^2$</td>
<td>m/s$^2$</td>
</tr>
<tr>
<td>TSA</td>
<td>1.40</td>
<td>1.37</td>
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</tr>
<tr>
<td>ESA</td>
<td>6.8</td>
<td>1.68</td>
<td>1.61</td>
</tr>
<tr>
<td>ESA</td>
<td>5.8</td>
<td>1.68</td>
<td>1.59</td>
</tr>
<tr>
<td>ESA</td>
<td>4.8</td>
<td>1.72</td>
<td>1.63</td>
</tr>
</tbody>
</table>
Figure 1. Liquefaction effects observed at San Carlo village after the 20 May 2012 Mw 5.9 earthquake: a, b) evidences of clay spread out on free field; c, d) evidence of clays spread out from building foundations causing damage to structures.
Figure 2. (a) Google Earth satellite view of the San Carlo village with trace of geological cross section and location of modelled soil columns. (b) geological map of the San Carlo village: 1) deposits of river channel; 2) deposits of river banks; 3) deposits of alluvial plain; 4) liquefaction
sand boil; 5) liquefaction trench; 6) borehole with samples; 7) borehole without samples; 8) Seismic DilatoMeter Test (SDMT); 9) Cone Penetration Test with piezocone (CPTU); 10) Seismic Cone Penetrometric Test with piezocone (SCPTU); 11) Seismic Cone Penetrometric Test (SCPT). (c) Geological cross section: 1) Holocene liquefiable deposits; 2) Holocene not liquefiable deposits; 3) Pleistocene liquefiable deposits; 4) Pleistocene non liquefiable deposits; 5) not classified Pleistocene deposits; 6) water table; 7) liquefaction sand boil; 8) liquefaction trench; 9) borehole; 10) CPTU or SCPT test; 11) modelled soil columns.

Figure 3. Spatial discretization of a horizontally layered soil. The seismic loading applied at the bedrock level is a deconvolved outcropping signal in terms of velocity.
Figure 4. Liquefaction front $r(S)$, where $r$ is the deviatoric stress ratio and $S$ is the state variable.

Figure 5. Normalized shear modulus reduction curves obtained using RC test results (markers) and fitted using the hyperbolic model (solid line), for soil samples S2-C2 (left) having reference shear strain $\gamma_{r_0} = 0.48\%$, S3-C3 (middle) having $\gamma_{r_0} = 0.39\%$ and S10-C3 (right) having $\gamma_{r_0} = 0.49\%$. 
Figure 6. Fitting of cyclic Consolidated Undrained triaxial test curves to calibrate liquefaction parameters, for S11-C1-2 soil sample in LS1 liquefiable soil layer.
Figure 7. Fitting of cyclic Consolidated Undrained triaxial test curves to calibrate liquefaction parameters, for S11-C3-3 soil sample in LS2 liquefiable soil layer.
Figure 8. Three components of the incident motion applied at the soil-bedrock interface in terms of acceleration. The peak acceleration is 2.54 m/s² in x-direction, 1.51 m/s² in y-direction and 0.33 m/s² in z-direction.
Figure 9. Profiles with depth of maximum shear strain and stress, horizontal velocity and acceleration, during the seismic event, for different water table depth and for TSA conditions in the C1 soil column. The horizontal dashed lines indicate the depth where the shear stress-strain loops are analyzed.
Figure 10. Hysteresis loops in C1 soil profile for different water table depth and for TSA conditions: (top) at 21.5 m in a non liquefiable soil layer; (bottom) at 13 m in LS2 liquefiable soil layer.
Figure 11. Profiles with depth of maximum shear strain and stress, horizontal velocity and acceleration, during the seismic event, for different water table depth and for TSA conditions in the C2 soil column. The horizontal dashed lines indicate the depth where the shear stress-strain loops are analyzed.
Figure 12. Hysteresis loops in C2 soil profile for different water table depth and for TSA conditions: at 9 m (top) and at 12.5 m (bottom) in LS2 liquefiable soil layer.
Figure 13. Profiles with depth of maximum shear strain and stress, horizontal velocity and acceleration, during the seismic event, for different water table depth and for TSA conditions in the C3 soil column. The horizontal dashed lines indicate the depth where the shear stress-strain loops are analyzed.
Figure 14. Hysteresis loops in C3 soil profile for different water table depth and for TSA conditions: (top) at 20.5 m in a non liquefiable soil layer; (bottom) at 12 m in LS1+LS2 liquefiable soil layer.
Figure 15. Profile with depth of minimum shear modulus (left), during the seismic event, with zoom in the first soil layers (middle) and profile with depth of the maximum reference shear strain, for different water table depth and for TSA conditions, in the C3 soil profile. The horizontal dashed lines indicate the depth where the shear stress-strain loops are analyzed.
Figure 16. Acceleration time history at the ground surface for C1 (left), C2 (middle) and C3 (right) soil profiles, in x-direction, for different water table depth and for TSA conditions.
Figure 17. Acceleration time history at the ground surface for C1 (left), C2 (middle) and C3 (right) soil profiles, in y-direction, for different water table depth and for TSA conditions.
Figure 18. Excess pore water pressure, normalized with respect to the average effective pressure, for different water table depth, in C1 (left), C2 (middle) and C3 (right) soil columns: (top) Profiles with depth of maximum value during the seismic event. The horizontal dashed lines indicate the depth where the time histories are analyzed. (bottom) Time history: at 21.5 m (LS1) in C1 soil profile, at 20.5 m (LS2) in C2 soil profile and at 4.5 m (LS1+LS2) in C3 soil profile, for $z_w = 3.7$ m, at 3.5 m for $z_w = 2.7$ m and $z_w = 1.7$ m.