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Interpreting the deformation phenomena of a levee damaged during the 2012 Emilia earthquake

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ABSTRACT

The May 20th, 2012 Emilia earthquake triggered significant fractures, deformations and liquefaction occurrences along a number of riverbanks located close to the epicentre area. One of the most severely damaged earth structures was a levee of an irrigation channel, where large, longitudinally-oriented ground cracks were observed along a 3 km stretch. The ground fissuring was apparently associated to a lateral spreading mechanism causing structural damage to the buildings settled on the bank crown.

An extensive study, including in-situ and laboratory investigations permitted a detailed definition of the geotechnical model and to back-figure the reference input motion at the deep bedrock. On such a basis, a dynamic effective stress analysis was carried out on a representative cross-section of the dyke showing that liquefaction occurred within the soil constituting the foundation of the levee. The results of the analysis allowed also for computing the permanent displacement along the critical sliding surface, which turned out to be compatible with the observed damage.

1. Introduction

Liquefaction phenomena observed during strong-motion earthquakes very often affected natural or artificial embankments. During the Hyogoken-Nambu earthquake (M = 7.2) which struck Kobe city (Japan) on January 17, 1995, loose riverbank sand deposits liquefied causing damages to about 9 km of levees stretches [1]. At most sites, sand boils were observed on the ground surface near the dykes [2].

The 1993 Kushiro-oki earthquake (M = 7.8), which struck the Hokkaido island in the North of Japan, induced significant dyke failures along the Kushiro river for a length of more than 10 km [3]. River dykes suffered extensive damage consisting of cracking, settlement, lateral spreading and slumping, all induced by liquefaction occurred in the body of the embankment (Fig. 1).

Due to the huge economic loss produced by the damage observed in the above mentioned cases, the analysis of seismic performance of levees was approached with increasing interest in Japan, as well as in United States. A systematic study of the dynamic response of characteristic levee cross sections for Central California was conducted by Miller and Roycroft [5], as well as by Athanasopoulos-Zekkos and coworkers ([6,7]), who proposed a simplified procedure for the evaluation of the seismic vulnerability of earthen levees. More recently Kwak et al. [8] proposed a simplified procedure to evaluate the fragility curves of levee systems, starting from the observed performance of those along the Shinano river, in Japan, heavily damaged by the 2004 M 6.6 Niigata-ken Chuetsu and 2007 M 6.6 Niigata-ken Chuetsu-oki earthquakes. The results of [8] were used for the fine-tuning of a more general guideline for multi-hazard analysis of river levees proposed by Zimmaro et al. [9].

In Italy, an increasing attention was focused on the protection and control of river dykes after the seismic sequence occurred in the Emilia-Romagna region since May to June 2012.

On May 20, 2012, a main shock of moment magnitude $M_W = 6.1$ caused severe damage on a large area in the Po river valley. This earthquake was mainly characterised by the extensive occurrence of liquefaction-induced damage, whereas previous evidences of soil liquefaction in the recent Italian seismic history are limited to few local manifestations, i.e. sand boils and surface deformations [10]. The most struck area is located in the south of the Po plain, in the foreland basin of two mountain chains, namely the Alps and the northern Apennines. A complex system of tectonic structures is underlying a thick sedimentary fill, so that the thrusts are generally buried [11,12]. The main tectonic structure is a buried ridge, known as 'Ferrara folds', which reaches its peak height, about 120 m below the ground surface, near the

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Fig. 1. Typical damage patterns induced by liquefaction on river embankments: evidences of (a) lateral spreading [3] and (b) slumping [4].

city of Ferrara [13-15].

The subsoil is characterised by alluvial deposits of different depositional environments, which consist of alternating layers of siltyclayey and sandy soils, mainly constituting ancient rivers banks. Most of the liquefaction evidences were observed along abandoned old channels and active inhabited levees [16]. One of the most damaged sites was an embankment along an irrigation channel known as 'Canale Diversivo di Burana', flowing through the small village of Scortichino, in the Municipality of Bondeno (Fig. 2).

An extensive study, including in-situ and laboratory investigations, was carried out in order to identify the possible causes of damage, as well as to suggest countermeasures for seismic risk mitigation. Liquefaction assessment of the embankment through empirical approaches and simplified dynamic analyses yielded ambiguous results [13].

Hereafter, dynamic effective stress analyses on a representative section of the dyke are described, in order to better understand the seismic performance of the embankment and the foundation soils during the earthquake.

The case study and the experimental investigations are briefly recalled in Section 2. The reference input motion at the deep seismic bedrock was obtained by de-convolving the surface ground motion recorded at a nearby station (Section 3). An accurate geotechnical model allowed for preliminary seepage and stability analyses (detailed in Section 4). Finally, a dynamic effective stress analysis on a reference soil column of the dyke was carried out in order to simulate the seismic response of the potentially liquefiable soils during the strong shaking (Section 5).

2. The case study

The "Canale Diversivo di Burana" was designed in 1884 with a dual aim. During the winter season, the channel collects rainwater from the countryside and several small villages through a dense network of secondary waterways, thereafter it delivers the rainwater in the Panaro river. On the contrary, the direction of the water flow is reversed during



Fig. 2. Scortichino bank stretch and location of section d-d'.

the summer season, when the Burana channel works as an irrigation canal for the surrounding plain. The waterway becomes a hanging channel along the Scortichino village, where an embankment was built. Over the centuries, houses and small economic activities settled on the crest of embankment, forming little hamlets along the channel where the cross-section of the embankment is wider (Fig. 2).

After the 2012 Emilia earthquake, longitudinally-oriented ground fissures were observed along a 3 km bank stretch, causing in turn severe structural damages to a large part of the approximately one hundred houses and economic activities built on the bank crown [13]. Many of the buildings were affected by several structural damages, such as masonry detachments and wall rotations, associated to the persistence of cracks along the floors and across the walls (Fig. 3).

In most cases, the fractures were 1–5 m long and following mostly the same alignments, while their horizontal opening was 2–3 cm wide on the average.

The damage survey carried out along and around the levee after the event revealed only spotty evidences of soil liquefaction, such as sand boils, detected at the land side at a quite significant distance from the toe of the dyke. Along the embankment crown, instead, no sand eruptions were observed; nevertheless, the longitudinal fissuring pattern described above was deemed as compatible with a lateral spreading mechanism (Fig. 1a).

In order to evaluate if it was necessary to relocate houses and any economic activities, the Emilia-Romagna regional authority committed to a task force of the Italian Geotechnical Society an in-depth study, aimed at identifying possible damage causes and relevant remedial measures, by evaluating the seismic response of the dyke during the 2012 earthquake sequence. A comprehensive geotechnical investigation was based on in-situ and laboratory tests, concentrated around the most damaged cross sections of the dyke [13]. The height of the dyke is variable from a minimum of 5 m to a maximum of 8 m at section d-d' (see Fig. 2), which corresponds also to the widest stretch of the embankment (about 55 m). In this study, the seismic performance of this representative section will be analysed in detail.

On the whole, the field investigation consisted of 5 boreholes reaching 20–50 m below the ground surface, from which undisturbed samples were extracted. Two boreholes were equipped with inclinometers down to 20 m depth, while the others were instrumented with piezometers. Moreover, penetration tests with piezocone (CPTU) and seismic dilatometer tests (SDMT) were carried out both at the crest and at the toe of the riverbank, down to a depth varying between 25 and 35 m. Finally, in-situ permeability tests were performed using the Lefranc method at depths of 8 and 15 m.

The laboratory experimental programme included a large variety of geotechnical tests for the determination of both static and dynamic properties of a number of 29 samples, such as undrained triaxial tests (TX-CIU), direct shear tests (DS), cyclic simple shear tests (CSS), double specimen direct simple shear tests (DSDSS), cyclic torsional shear tests (CTS), and resonant column tests (RC). All the details of the extensive



Fig. 3. Typical longitudinal cracks observed along the bank crown (a) and across a building (b) [13].

field and laboratory investigations, as well as the interpretation of the results, can be found in Tonni et al. [13].

3. Reconstruction of the reference input motion

The main shock of the Emilia 2012 earthquake sequence occurred on May 20, 2012 at 02:03:53 UTC time. The MRN station of the Italian strong-motion network (RAN), located in Mirandola town [17], is the closest to the epicentre (Fig. 4a) and recorded a peak ground acceleration, PGA, as high as 0.273 g.

The record cannot be used directly as a reference input motion in a seismic response analysis, because the station is located on a deep soft subsoil, with an equivalent shear wave velocity $V_{S,30} = 208 \text{ m/s}$, i.e. a Class C site according to Eurocode 8 (EC8) [18]. As previously mentioned, the 2012 seismic sequence affected an alluvial plain with a significant depth of the seismic bedrock, so that the closest stations located on a rock outcrop lie too far from the epicentre. To overcome this problem, the acceleration record at MRN station was deconvolved to the bedrock (Section 3.1); thereafter, the deconvolved outcrop

motion was scaled to account for the lower epicentral distance of the Scortichino site (Fig. 4b), as it will be detailed in Section 3.2.

The EW component of the recorded mainshock has been considered in the following, since it is characterised by the highest *PGA* value (Fig. 5c).

3.1. Deconvolution of surface motion

The deconvolution procedure requires the definition of the subsoil model under the recording station. The MRN station is located along the Napoli street in the Mirandola municipality, very close ($\approx 100 \text{ m}$) to the site where a downhole seismic array was deployed after the earthquake [12]. During the installation of the array, a cross-hole test was carried out until 125 m depth. Figs. 5a and 5b respectively illustrate the soil layering and the shear wave velocity profile (grey line) as obtained from the cross-hole test. The stratigraphy consists of an alternating sequence of silty (B), sandy (A200, A400) and clay (C) layers. At significant depth from the surface, alternations of sand and clay have been modelled as a single material (AL).



Fig. 4. Location of the MRN station with respect to (a) the Italian strong-motion network [17] and (b) the case study (the fault box is drawn in red; the triangles with different colours represent different ranges of PGA recorded).



Fig. 5. (a) Subsoil profile at MRN station; (b) shear wave velocity and peak acceleration profile;(c) recorded EW acceleration time history, (d) normalized shear modulus and damping curves; and (e) deconvolved outcrop motion in terms of acceleration time history.

This profile was adopted for defining the variation with depth of the small strain stiffness along the layered subsoil model under the recording station. Since no site-specific laboratory data were available, the shear modulus reduction and damping curves (Fig. 5d) were assumed to be the same as those measured by the cyclic and dynamic laboratory tests on the soil samples taken at Scortichino [13].

The conventional deconvolution procedure consists of assigning a recorded ground motion at the surface of a one-dimensional soil column and using a linear equivalent analysis to back-calculate the acceleration time history at the bedrock. The standard, as well an alternative deconvolution procedure suggested by Silva [19] for soft sites, have been adopted in the following analyses.

In detail, Silva measured the coherence between the simulated propagated motion and that recorded at the surface in order to provide an estimate of goodness of fit. The results showed that the coherence between the recorded surface motion and that analytically propagated to surface drops off at the frequency 15 Hz. For this reason, he suggested applying a low pass (LP) filter at 15 Hz to the recorded surface motion to be used for the de-convolution analysis. Moreover, from the coherence analysis it appeared that at least the 75% of the total energy is related to the propagation of normally incident shear waves. The remaining energy may be due to scattered waves and perhaps to Pwaves. This has significant implications on the non-linear behaviour of the soils.

In particular, attempting to deconvolve the total surface motion as vertically propagating shear waves may result in too much energy predicted at depth, leading to excessive estimates of modulus reduction and mobilised damping. The overall effect is the overestimation of the motion at depth that is required to produce the total observed surface motion [19].

The guidelines for equivalent-linear de-convolution proposed by Silva [19] and summarised by Markham et al. [20] in the following steps were adopted in this study:

- a 4th order low pass (LP) Butterworth filter at 15 Hz was applied to the recorded surface motion to be used for the deconvolution analysis (Fig. 5c); the acceleration time history was scaled by a factor of 0.87;
- 2. the filtered and scaled motion from step 1 was assigned as input motion at the surface of the 1D soil column;
- the motion at the layer of interest below the surface was computed via an equivalent linear seismic response analysis;
- 4. the shear modulus and damping profiles mobilised at the end of the analysis were computed;
- 5. the deconvolution process was carried out again performing a linear analysis of the 1D column characterised with the mobilised values of stiffness and damping, previously computed (step 4), and applying the LP filtered (15 Hz) full surface motion (i.e. not scaled by 0.87) at the top of the column to obtain the final outcropping de-convolved motion.

In this study, the EERA [21] code was utilized to perform both deconvolution analyses. The de-convolved reference input motion obtained through the alternative procedure is shown in Fig. 5e. It is worth noting that the outcrop accelerogram was about the same as that resulting from the conventional approach, with a *PGA* equal to about 0.30 g in both cases.

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Fig. 6. (a) Comparison between ground motion parameters of the deconvolved (MRN-EW) and scaled (Scortichino) accelerograms with Italian GMPE predictions: (a) *PGA* and (b) *PGV* vs Bindi et al. [23];(c) T_m and (d) D_{5-95} vs Tropeano et al. [24] GMPEs. (For interpretation of the references to color in this figure legend, the reader is referred to the web version of this article).

3.2. Scaling of the deconvolved outcrop motion

Due to the difference between the epicentral distance of the MRN station and that of the dyke site (Fig. 4b), the effect of ground motion attenuation was duly accounted for.

Similarly to previous numerical studies relevant to the same area [22], the ground motion prediction equation (GMPE) based on the Italian strong-motion database was adopted [23]. The procedure consisted of scaling the ground motion amplitude for the Joyner-Boore distance, R_{JB} , by means of the attenuation law relevant to the magnitude ($M_w = 6.1$) and faulting mechanism (thrust) of the seismic event.

The fault projection on the ground surface (Fig. 4b) allows for computing the Joyner-Boore distance of the MRN station, R_{JB} = 4.34 km; instead, the Scortichino dyke is located inside the surface projection of the fault (R_{JB} = 0.1 km as default value). Therefore, at this site the ground motion amplitude at rock outcrop must be expected as higher than that back-figured through de-convolution at the MRN station.

The black line in Fig. 6a is the median attenuation law, while the grey-hatched area represents the range of prediction corresponding to the 16th to the 84th percentile. At MRN station, the *PGA* of the deconvolved outcrop motion (plotted with a blue diamond symbol) falls slightly above the median prediction. The GMPE was therefore scaled so that to reproduce this specific *PGA* value at the same source-site distance (dashed blue line in Fig. 6a), corresponding to the 64th percentile. The latter was supposed as characterizing even the value of *PGA* expected at the site of Scortichino (red empty diamond), assuming the statistical error as isotropic and independent of the distance in the

epicentral area. The value of *PGA* (0.344 g) predicted at Scortichino site was therefore used to scale the deconvolved motion (Fig. 5e).

In Fig. 6b, the *PGV* value of de-convolved MRN accelerogram is compared with the prediction of the relevant GMPE [23]: it corresponds to a higher percentile with respect to *PGA*, showing the same statistical error as *PGV* evaluated from the scaled accelerogram at Scortichino.

Fig. 6c and d respectively show the comparison of mean period, T_m , and significant duration, D_{5-95} , of the deconvolved and scaled accelerograms with the GMPEs proposed for Italian seismicity by Tropeano et al. [24]. The deconvolved signal at MRN has a significant duration close to the median value predicted by the GMPE. The median period, on the other hand, is close to the 16th percentile, implying a signal signature with a prevalence of high frequencies, which excludes significant impulsive near-source effects. Note that the two sites are located at R_{JB} distances falling in ranges where the gradients of the GMPEs for both parameters are negligible. Such a poor variability of the mean period and significant duration suggested that further scaling, e.g. also in terms of frequency, of the de-convolved ground motion was not necessary.

4. Basic geotechnical model and preliminary seepage and stability analyses

Fig. 7 shows the reference cross-section d-d' of the dyke, as obtained by analysing the results of boreholes and field tests. The core of the dyke and its foundation soil consist of a silty sand, which rests upon a thick deposit of alluvial sands, interbedded by clay. Material parameters assumed for the different stratigraphic units are reported in



Fig. 7. Stratigraphic model of the cross section d – d' with piezometric surface (blue line), critical sliding mass (green shade) and reference soil column (red line). (For interpretation of the references to color in this figure legend, the reader is referred to the web version of this article).

Table 1

Soil properties used for seepage and stability analyses.

Soil	Hydraulic conductivity k (m/s)	Friction angle $\varphi'(^\circ)$	Coefficient of pressure at rest <i>K</i> ₀	Poisson's ratio ν
dyke	1.10^{-6}	33	0.44	0.306
silty sand	1.10^{-6}	33	0.44	0.306
clay	1.10^{-8}	25.5	0.59	0.371
sand	3.10^{-5}	38	0.4	0.286

Table 1.

Seepage analysis with the code SEEP/W [25] was used to define the groundwater level within the embankment section, consistent with the piezometric measurements carried out in the deep sands (Fig. 7).

Due to the high degree of saturation of the undisturbed samples (> 95%), taken at a shallow depth from the ground surface, the seepage analysis was carried out using the saturated hydraulic conductivity of soils.

The hydraulic conductivity of the clay deposit (C) was back-figured from the interpretation of the pore pressure dissipation curves recorded during CPTU tests, while those of the silty sand and sand layers were measured by the LeFranc permeability tests carried out at 8 m and 15 m depths respectively. The procedures and results of the above mentioned tests are detailed in the report by [26].

The boundary conditions for the seepage analysis were assigned as the level of water table in the channel and the hydrostatic condition at the landside of the embankment, where the piezometric measures detected the ground water table at 7.7 m below the ground surface [13].

The trend of the piezometric surface inside the embankment, as obtained by the seepage analysis (reported in Fig. 7 with a blue line), highlights that the slope stability at the riverside of the dyke could be strongly reduced by the increase of pore pressure in seismic conditions [13]. On this side of the bank, saturated silty sands (potentially lique-fiable) are overlaid by a non-liquefiable crust of significant thickness, at least 7 m thick in correspondence of the crest. Such a high thickness could justify the occurrence of liquefaction in the sandy layer without any evidence of sand boil at surface, as confirmed by the relationship suggested by [27].

For evaluating the seismic slope stability at the riverside of the dyke, a yield acceleration of 0.175 g was computed adopting the Morgenstern-Price [28] method and using drained shear strength parameters in the analysis. The potentially sliding mass obtained by pseudo-static analysis is drawn in light green in Fig. 7.

5. Model and procedure for the dynamic analyses

The dynamic numerical analyses were carried out in two stages, by adopting the equivalent slope approach as suggested by Blake et al. [29]. A representative soil column of the slope was chosen (red dashed line in Fig. 7) as that corresponding to the maximum depth of the critical sliding surface [30]. Along this soil column, a one-dimensional non-linear seismic response analysis in effective stress was first carried out, with the twofold purpose of:

1. evaluating the excess pore pressure triggered by the earthquake;

2. defining an equivalent acceleration time history.

This latter accelerogram was thereafter used for predicting the permanent displacement, according to the conventional 'uncoupled procedure' first suggested by Bray & Rathje [31] (see also [24]).

On the basis of geological studies [15], in the subsoil model the seismic bedrock was located at 115 m depth (Fig. 8a), i.e. at about the same depth as that detected by the deep investigations at Mirandola (see Fig. 5a). The shear wave velocity profile (Fig. 8b) was defined by integrating the results of the SDMTs (reaching at most 50 m of depth) with the measurements of two Cross-Hole tests carried out in the towns of Mirandola and Medolla, which reached 130 m below the ground surface [32].

The deepest stratigraphy essentially consists of a thick sand layer



Fig. 8. (a) Subsoil layering and (b) shear wave velocity profile at the d-d' section of the dyke.

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resting on an alternation of centimetric layers of sands and silts (Fig. 8a).

The curves describing the variation with shear strain of the normalized shear modulus and the damping ratio, adopted to simulate the non-linear behaviour of the different soil layers, were the same as those used for the deconvolution analysis, shown in Fig. 5d. The characterization of the shallow sand (i.e. until a depth of 35 m, yellow layer in Fig. 8a) was based on DSDSS tests performed at a confining pressure between 180 and 200 kPa; the tests executed at a confining pressure of 400 kPa were instead used to characterize the deeper sands. For the deepest layer, constituted by alternations of sand and clay, average curves between those of the clay and the deep sands were adopted.

The shear modulus reduction curves were analytically fitted by using the 'Modified Kondner and Zelasko' (MKZ) model [33], according to the procedure for strength compatibility proposed by Gingery and Elgamal [34], in order to better predict the soil behaviour at large strains up to failure [35,36].

The build-up of excess pore water pressure in the saturated soils was simulated through a simplified model, described in detail in [37,38]. The model formulation is based on the definition of a 'damage parameter', which permits to synthetically express the seismic demand relevant to an irregular time-history of shear stress, and to compare it to the cyclic strength of liquefiable soils, as measured in stress-controlled cyclic laboratory tests [38]; as a result, the pore pressure ratio (i.e. the excess pore pressure normalized by the initial effective confining stress) can be straightforward computed as a function of the cumulated damage.

Fig. 9 shows the experimental results of cyclic simple shear tests on undisturbed samples of silty sand and clean sand [13]. The number of cycles to liquefaction, N_L , was established assuming that liquefaction occurs at a pore pressure ratio $r_u = 0.90$.

The model adopted in this study describes the cyclic resistance curve as follows (Fig. 9a):

$$\frac{(CSR - CSR_t)}{(CSR_r - CSR_t)} = \left(\frac{N_r}{N_L}\right)^{\frac{1}{\alpha}}$$
(1)

where:

- CSR is the cyclic stress ratio, i.e. the shear stress amplitude normalized by the initial effective confining pressure;
- N_L is the number of cycles at liquefaction;
- (N_r, CSR_r) is a reference point on the cyclic resistance curve;
- the exponent α describes the dependency of the cyclic resistance from the number of cycles;
- *CSR_t* represents the horizontal asymptote of the curve.

Since the lower bound, *CSR*_t, of the cyclic resistance curve was not clearly defined by the experimental data, the corresponding shear stress

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Table 2

Parameters of the pore pressure model for the different soils.

Soil	α	CSR_t	CSR _r	а	b	с	d
Silty sand	1.85	0.078	0.18	0.81	0.54	0.09	4.00
Sand	1.97	0.08	0.16	0.54	0.50	0.36	4.00

amplitude was estimated from the stiffness-strain relationship measured in resonant column tests as that corresponding to the volumetric threshold strain [39].

The parameter α was finally determined as the slope of the cyclic resistance curve in a semi-logarithmic plot. Moreover, the pore pressure model expresses the pore pressure ratio, r_u , as a function of the normalized number of cycles, N/N_L (Fig. 9b):

$$r_u = a \left(\frac{N}{N_L}\right)^b + c \left(\frac{N}{N_L}\right)^d \tag{2}$$

where a, b, c and d are curve-fitting parameters. The pore pressure parameters calibrated for the two sandy soil deposits are reported in Table 2.

The dissipation and re-distribution of pore pressure, modelled using the one-dimensional theory of consolidation, were also taken into account in the analysis.

The ground level and the top of the bedrock, constituted by a fractured calcareous rock, were assumed as free drainage surfaces. The one-dimensional consolidation coefficient, c_v , was defined as a function of the hydraulic conductivity and of the constrained modulus, E_{oeds} expressed as follows:

$$E_{oed} = \frac{2G_0(1-\nu)}{(1-2\nu)}$$
(3)

where ν is the Poisson's ratio, computed from the coefficient of pressure at rest (Table 2). This latter was obtained from the horizontal stress index, K_D , resulting from the dilatometer test in the clay layer, while it was empirically correlated to the friction angle measured in the TX-CIU tests (cf. Table 1), according to the Jaky [40] relationship, for the sandy and silty sand layers.

The build-up and dissipation models were implemented in the code SCOSSA [41], in order to carry out 1D coupled dynamic analyses in terms of effective stresses [42]. The numerical procedure is based on a lumped parameter discretization of a soil column and time domain integration of the dynamic equilibrium equations. The non-linear soil behaviour is modelled by a hysteretic stress-strain relationship based on the MKZ model corrected through the above mentioned procedure by [33], with generalised Masing rules to reproduce loading-unloading-reloading cycles [41].



Fig. 9. Cyclic resistance curves (a) and pore water pressure relationship (b) for clean sand and silty sand.

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Fig. 10. Results of the effective stress analysis in terms of profiles of peak (a) acceleration, (b) shear strain, (c) shear stress and (d) pore pressure ratio.

6. Results of the dynamic analyses

Fig. 10a-b-c-d report the results of the 1D effective stress analysis in terms of profiles of maximum acceleration, shear strain, shear stress and pore pressure ratio. As shown in Fig. 10d, liquefaction is expected to occur between 3 and 12 m below the ground surface, within the silty sand layer and the shallowest part of the underlying sand deposit. The same figure shows the thickness of the liquefiable soil as resulting from the simplified liquefaction analysis [11] based on the CPTU tests (hatched grey area). Note that the simplified analysis predicts the occurrence of liquefaction exclusively in the silty sand deposit under the dyke.

Peak shear strains even higher than 2.5% (a conventional liquefaction threshold level) were reached at about 12 m under the ground level (Fig. 10b), due to the accumulation of plastic strains in the τ - γ cycles of the liquefiable soils.

Right above the spikes in the acceleration and strain profiles (Fig. 10a-b), the results show a significant attenuation of motion approaching the surface; this effect can be attributed to the degradation of stiffness and the increase of damping of the liquefied soil layer, which works like a kind of seismic isolation device, by limiting the

acceleration although increasing the displacement.

Fig. 11a shows the time history of shear stress and excess pore pressure at the depth of the critical sliding surface, which is located in the silty sand layer (i.e. 2.2 m under the surface of the reference soil column). Every time the shear stress overcomes the threshold value, τ_t (black dashed line in Fig. 11a), an excess pore pressure increment is generated. The dissipation process starts after the most critical stage of the motion (~16 s) and it continues even after the end of the seismic shaking; nevertheless, the dissipation rate appears rather high, being the sliding surface very close (0.5 m) to the groundwater level.

A sliding block analysis was carried out to evaluate the permanent displacement amplitude, by assuming as input motion the 'equivalent accelerogram', computed from the resultant of the inertia forces in the soil column overlying the depth of the critical sliding surface. In detail, the equivalent accelerogram, $a_{eq}(t)$, was computed as the ratio between the shear stress time history at the depth of the sliding surface, $\tau(t, z = 2.2 \text{ m})$, and the vertical stress at the bottom of the unstable mass, $\sigma_{\nu}(z = 2.2 \text{ m})$, as follows:

(4)



 $a_{eq}(t) = \frac{\tau(t, z = 2.2\mathrm{m})}{\sigma_v(z = 2.2\mathrm{m})}\mathrm{g}$

Fig. 11. (a) Shear stress and excess pore pressure ratio, (b) equivalent acceleration and displacement time histories at the depth of the critical sliding surface (2.2 m) for the reference soil column.

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Fig. 11b shows the time history of displacement, δ , resulting from the rigid block analysis: accumulation of displacement occurs when the acceleration overcomes the yield acceleration, a_{y} .

It can be noted that the final displacement is 1.6 cm, which is compatible with the width of the cracks observed along the dyke.

7. Conclusions and perspectives

The paper presents the results of numerical simulations addressed to evaluate the seismic performance of a river bank severely damaged after the 2012 Emilia earthquake. The multi-stage procedure adopted, including seepage and stability as well as dynamic seismic response and displacement analyses, could rely on an accurate geotechnical characterization, supported by comprehensive and high quality field and laboratory investigations.

Such data were also helpful to reconstruct the reference input motion at the deep seismic bedrock, by applying both a standard and a non-conventional deconvolution procedure to a seismic record collected by a nearby station, lying on a comparable subsoil layering.

A 1D dynamic effective stress analysis was carried out on a soil column, assumed as representative of the seismic behaviour of the riverside slope of the dyke. The analysis was performed using a code developed by the authors, in which a recently formulated simplified model was implemented to predict pore pressure build-up and dissipation.

The results demonstrate that liquefaction occurred throughout a soil thickness of about 10 m across two layers of silty sand and clean sand constituting the foundation soil of the dyke; excess pore pressure are also predicted to develop in the clean sand deposit down to 60 m under the ground surface. The outcomes of these simulations straighten out the controversial results of liquefaction assessments obtained applying the conventional empirical methods, based on different in-situ measurements [13].

Finally, the permanent displacement along the critical sliding surface was computed by following a kind of uncoupled approach, based on the application of the rigid block model, using the equivalent acceleration time history obtained with the effective stress analysis.

Several limitations of the overall approach followed in this study are acknowledged and claim for further refinements. First of all, significant uncertainties still affect the definition of the reference input motion, due the lack of seismic records not only at the levee site, but also in the surroundings. An alternative to the procedure adopted herein, based on de-convolution and scaling of a nearby record through attenuation laws, could be represented by the generation of a synthetic bedrock motion through physics-based modelling of source and propagation mechanisms [43].

A hybrid numerical procedure was preferred in this case to more complex and comprehensive approaches for modelling. Notwithstanding its inherent simplicity in terms of geometrical aspects (one-dimensional seismic response analysis) and constitutive modelling (rigid block model), it was possible to reproduce the pore pressure build-up as well as to predict a permanent displacement value compatible with the observed damage.

Further developments in the performance of the SCOSSA code are in progress, such as the combination of the pore pressure model with the 'stick-slip' procedure for the numerical integration [41], aiming at a straightforward and reliable prediction of the permanent displacement, along with the computation of acceleration and pore pressure.

Future studies can be also specifically addressed to evaluate the effect of aftershocks on the development of progressive liquefaction phenomena. Indeed, as shown by Sinatra and Foti [22], the excess pore pressure caused by the mainshock at the nearby villages of San Carlo and Mirabello was likely fully retained when two subsequent after-shocks occurred about two and three minutes after the first shaking. This mechanism, caused by typical stratigraphic conditions where liquefiable layers are confined on top and at the bottom by low

permeability layers, could have also affected some stretches of the Scortichino river bank. It is in the Authors' expectations that the peculiarities of the computer code adopted in this study, i.e. its simplicity of use due to one-dimensional scheme and the consequent reduction of the time of computation, can make it as particularly suitable to predict the seismic performance of embankment-soil systems under sequential seismic events.

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