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Feasibility study of a loss-driven earthquake early warning and rapid response systems for tunnels of the Italian high-speed railway network

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ABSTRACT

Linear infrastructures have strategic importance and impact on the social and economic conditions of many countries, hence the seismic risk management of existing and new designed ones is a crucial issue in earthquake-prone areas. High-speed and high capacity railways are an example of infrastructures that assume increasing importance in developed countries, since they permit rapid transit of people and freight.

Due to the seismicity of the country, the case of the high-speed railways Italian network appears suitable for assessing the feasibility of a loss-driven earthquake early-warning system based on the real-time estimation of the expected damage probability and lead-time. Among the several subsystems that compose the network, the paper focuses on tunnels, since they are largely present along the route of the existing high-speed lines and of the new ones currently under design.

This work describes a procedure that exploits the disaggregation of the seismic hazard to define sets of virtual seismic sources potentially affecting railway's tunnels. Hence, the probability of seismic damage to tunnel structures and the time available for implementing real-time mitigation procedures can be calculated. Such a procedure is applied to two tunnels of the high-speed system with different structural layout. The procedure suggests that for the considered tunnels the best option for undertaking seismic risk mitigation measures would be an on-site threshold–based early-warning system. However, the foreseen probability of structural damage to the tunnel lining is low in both cases.

The proposed methodology can be easily generalized to different targets to design the optimal configuration of an earthquake early warning system, and applied to control, manage and maintain the tunnel structures along the high-speed railway network.

1. Introduction

Over the last decades, many countries faced an increasingly growing demand of urbanization and were thus forced to exploit the underground space in order to develop their 'physical inter-connectivity'. A well-fitting example of physic inter-connectivity among urban areas is the high-speed rail network (defined according with the more recent technical specification of the European Committee as the rail network where trains travel at a speed $V_{HS} \geq 250$ km/h, hereinafter HSR), which widely spreads in many parts of the world with hundreds of kilometers underground.

Nowadays, high-speed rail systems cover a large slice of the mass transportation in the world, shortening distances and reducing travelling time. Such advantages have, definitely, an important economic impact on many countries that continue to invest in this sector, but also social benefits within a highly globalized context.

The HSR system has achieved the higher performance in Japan, where the Tokaido Shinkansen has been the first high speed line in the world, followed by Europe and America respectively. The European HSR system developed later than the Japanese one, but nowadays it counts the 60% of the worldwide network, with the ambitious goal, still under completion, to connect the entire continent by the Trans-European Networks – Transport (TEN-T). Italy is fully integrated in the TEN-T, both in terms of achieved effective velocity (i.e., 300 km/h) and of national rail network coverage (i.e., about equal to 8%; Table 1).

Considering that Italy is a highly seismic country, the effects of ground shaking induced by earthquakes on the HSR systems are a matter of concern for the maintenance of existing railways and the design of the new ones. Undoubtedly, the major concern for the railways companies is the potential derailment of a high-speed train due to ground shaking, since it would cause severe injuries, casualties and economic loss. However, damages induced by an earthquake to the railway infrastructure (i.e. embankments, bridges, tunnels) are also important considering the direct economic losses produced by the

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Table 1
Worldwide High Speed rail classification (extrapolated from https://www.goeuro.it/treni/alta-velocita).

	Country -	*Record velocity Km/h*	*Effective velocity Km/h*	*Network coverage %*	*Population coverage %*
1	*Japan* SC Maglev	603	320	12.23	36.55
2	*France* TGV	575	320	6.79	12.69
3	*China* Shanghai Maglev	501	350	29.22	10.70
4	*Corea* KTX	421	300	1.62	44.67
5	*Spain* AVE	404	320	20.05	20.51
6	*Italy* Frecciarossa 1000	400	300	7.91	18.47
7	*Germany* ICE	368	320	4.75	18.28

service disruption and any possible indirect effect (such as train derailment).

Focusing on the effect of ground shaking on tunnels, that are largely present along the route of the Italian high-speed railways network, this work describes the seismic hazard and the seismic vulnerability of the tunnel lining. This work is motivated by the observation that the high-speed rail network crosses complex seismogenic regions with a moderate to high seismic hazard (Fig. 1), and therefore, tunnels of the railways network might undergo important level of acceleration, and accordingly damages.

Among the various aspects that can be considered during a performance analysis of a tunnel (i.e. structure; geological, geotechnical and hydrological uncertainties; localized phenomena), the seismic vulnerability is certainly one of the most important in areas exposed to high seismic hazard. Past earthquakes, indeed, revealed that important seismic events can damage the tunnel structure and limit or temporarily inhibit its functionality [1–5]. The available data are mostly referred to tunnels excavated with traditional methods. These studies show how the deformation induced by the earthquake to the tunnel can produce different crack patterns in the lining (in longitudinal, transverse or generally inclined direction) according with the complex seismic soil-tunnel interaction mechanism, which in turn depends on the lining and soil stiffness properties, the soil-structure interface behaviour, the maximum soil acceleration and the direction of propagation of the seismic event with respect to the structure.

From an engineering point of view, the seismic risk management of tunnels is accomplished by computing the probability of damage (P_f) due to seismic actions. This is calculated as the convolution of the seismic vulnerability (V) and the seismic hazard of the specific site (H). Furthermore, the whole seismic risk assessment must include also the exposure of the tunnel itself, as well as that of the high-speed trains and their customers, that is variable with time.

A widespread methodology for seismic vulnerability assessment for single structures or a class of them makes use of fragility curves computed for increasing seismic hazard level. The combination of fragility curves and seismic hazard derived scenarios can then be used to study the tunnel performance under seismic actions, also in real-time [6].

Taking in to account the importance of developing strategies suitable for the real-time mitigation of seismic risk for railways tunnels in

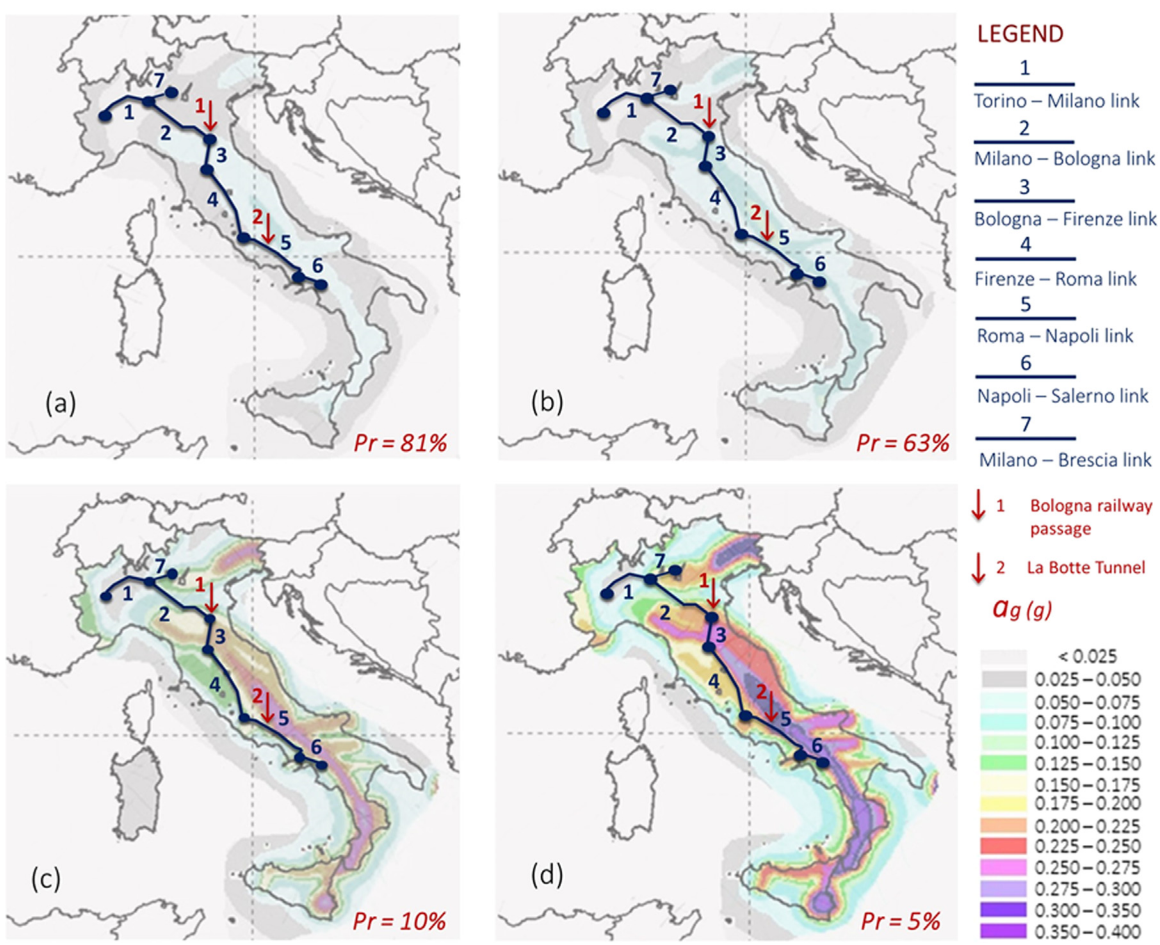

Fig. 1. Italian high speed rail system (operational) combined with the hazard maps for a probability of exceedance, PR, equal to (a) 81% (return period TR = 30 y), (b) 63% (return period TR = 50 y), (c) 10% (return period TR = 475 y), and (d) 5% (return period TR = 975 y). (Hazard maps extracted from INGV, http://zonesismiche.mi.ingv.it).

every country prone to seismic hazard, in this work we propose a novel approach for the feasibility analysis of a loss-driven earthquake early warning (loss-EEW) and rapid response system for tunnels. The study is focused, as an example, on two significant sites of the Italian HSR network.

The proposed procedure overcomes the problem of lack of accelerometric data (i.e., waveforms) from past earthquakes that often hampers feasibility studies by exploiting the disaggregation of the seismic hazard to define for each of the targets (i.e., the tunnels) a set of virtual seismic sources, characterized in terms of epicentral distance from tunnels and magnitude. Hence, in this sense, the approach can be easily generalized and applied to different areas. The performance of the loss-EEW system is assessed in terms of lead-time and expected damage probability.

2. Background

The structure susceptibility to be damaged during a seismic event is indicated herewith as vulnerability, while the natural or man-made changes introduced into a system capable of affecting adversely its current or future performance is indicated as damage [7,8].

Estimating the seismic vulnerability of a structure is a crucial piece of information for evaluating the probability of a structural failure and to take decisions aiming to mitigate the seismic risk. In the case of tunnels, their vulnerability to ground shaking is affected by: (i) the interaction of the structure with the surrounding soil, which depends on the relative soil-structure stiffness; (ii) the interface conditions at the contact between the soil and the tunnel [9–12]; (iii) the soil dynamic behaviour; the tunnel lining technology [13]; and finally, of course, (iv) the ground motion level.

The fragility functions provide a link between the seismic hazard assessment at a specific site and the corresponding effects on the specific structure in terms of reached or exceeded probability of damage, given a level of ground motion intensity measure (IM). The performance levels of a structure are defined through damage thresholds, called limit states, that define the boundaries between two different damage conditions, called damage states (DSs). The fragility functions can be generally grouped in two main groups: analytical and empirical curves. Analytical fragility functions allow estimating the damage distribution through analyses of the structural models subjected to increasing seismic loads. Empirical fragility curves are based on observation collected during the past years by post-earthquake surveys on structures and the assessment of their damages. Empirical methods have the advantage to be defined on real observed data, but the disadvantage to be usually based on low magnitude event with limited damages, in comparison to the analytical methods. For this reason, empirical methods are thus considered less reliable than analytical ones for estimating suitable fragility curves for higher magnitude events. It is also worth noting, however, that in literature only a limited number of analytical and empirical approaches for fragility analysis for tunnels exists.

Analytical fragility curves have been developed for tunnels by Salmon et al. [14], Argyroudis & Pitilakis [15], Andreotti & Lai [16], Argyroudis et al. [17], Kiani et al. [18] and Fabozzi et al. [19]; and generally they have been derived for different ground conditions, corresponding to the site classification according to Euro-code EC8, and for a specific tunnel lining geometry and technology. For instance, Salmon et al. [14], proposed analytical fragility curves for bored and cut-and-cover tunnels of the BART project as a function of peak ground acceleration PGA (ground shaking) and permanent ground deformation PGD (fault offset). Argyroudis and Pitilakis [15], have studied the case of a circular continuous tunnel lining for three different ground categories (according to EC8), defining different level of damage states, DSs (i.e., minimum, moderate and extensive), as a function of PGA, for a specific damage index, DI. The latter is defined by the authors as the ratio between the demand and the resistant bending capacity of the concrete section. Andreotti & Lai [16] proposed analytical fragility curves for deep tunnels defining a damage index, DI, as the number of the activated plastic hinges in the tunnel liner. Argyroudis et al. [17], developed further fragility curves highlighting the important role of the soil conditions and the corrosion effects on the vulnerability of the tunnel structures. Kiani et al. [18], proposed numerical fragility curves for segmental tunnel lining crossing faults, based on the results of experimental centrifuge tests. Fabozzi et al. [19] have adapted the study of Argyroudis and Pitilakis [15] to the case of a circular segmental lining, defining three DSs (i.e., minimum, moderate and extensive) for a DI defined as the ratio between the demand and the rotational capacity of the longitudinal joint.

Empirical fragility curves for tunnels were proposed by ALA [20], NIBS [21] and Corigliano [5]. ALA [20], provided empirical fragility curves as functions of peak ground acceleration (PGA) at soil surface, considering soil conditions and lining type for three DSs (i.e., minimum, moderate and heavy); the HAZUS approach, NIBS [21], is based on expert judgment and a limited empirical data set by Dowding & Rozen [1] and Owen & Scholl [2] and provide fragility functions for both ground shaking (input motion parameter: PGA) and earthquake-induced ground failure (input motion parameter: PGD); Corigliano [5] presented fragility curves for deep tunnels as functions of peak ground velocity (PGV) for slight and moderate damage.

Overall, empirical fragility functions do not differentiate neither for the geologic media, nor for the type of lining. Moreover, they neglect the fact that continuous and segmental lining exhibit a different response during earthquake [22]. It is worth noting that, as observed for example during the Kobe earthquake of 1995 [23], the connections between adjacent segments are the most vulnerable points of the tunnel structure.

In a context where strategic structures are settled in hazard-prone areas and deserve protection against harmful seismic events, Earthquake Early Warning Systems (EEWS) can be a very effective solution [24]. Nowadays, indeed, the theoretical and methodological advances in real-time analysis of seismic data accompanied by a rapid improvement in telemetry and computer technology allows the characterization of earthquakes within a few seconds from their occurrence (i.e., epicentral location, magnitude and ground motion level at target sites are available within 2–3 s from the moment when early P-waves signal are recorded by seismic stations [25]). EEWS are typically classified in two approaches: regional (or network based) and on-site (or stand-alone) systems. Regional systems exploit dense strong motion networks surrounding faults known as potential seismic threats [26]. On the other hand, onsite systems exploit one or more seismic sensors installed at the site to secure, where empirical amplitude-scaling relationships between early P-wave signals and S-waves are used to anticipate the shaking [27,28]. A key parameter for implementing protection actions against seismic risk is the lead-time. This is defined: for regional EEW system as the travel time difference between the arrival of the first S waves (i.e., relatively slow seismic waves carrying much of the energy radiated from the source, and hence having a high destructive potential) at the target site and the early P wave (i.e., fast and small amplitude seismic waves) recorded at the source area, after accounting for the necessary computation and data transmission times; and for the on-site EEW system as the travel-time difference between S- and P-waves at the target itself. An exhaustive review of the concepts, methods, and physical basis of EEWS has been presented by [26].

Very few operational EEWS for railway network exist worldwide yet. The best-known of them is the one developed by the Japanese Railway to slow down or stop trains before seismic shaking affected trains running at high speed along Shinkansen network [29]. The Great East Japan Earthquake of 2011, despite the success of the EEWS operating on the Shinkansen network, highlighted the importance of extending EEWS towards real-time monitoring and loss assessment system for structures linked to the rail-tracks (e.g., tunnels, platforms, station building, electric poles; MTI Report 12–37; http://transweb.sjsu.edu/

project/1225.html). Recently, Picozzi et al. [30] and Pittore et al. [31] proposed to exploit real-time earthquake location and size estimates from EEWS to extract seismic actions from a portfolio of precomputed scenarios and for generating risk scenarios in real-time. In the case of building monitoring, the real-time accelerometric stream can be used into a performance based EEW approach as proposed by Iervolino [6]. Bindi et al. [32] showed an application to buildings in Central Asia where, combining the fragility curves with the ground motion predicted for S-wave from the P-wave parameters recorded at a sensor at the building ground level, the probabilities of different damage levels were predicted.

3. Case of study

3.1. The Italian high-speed rail system

The Italian High-Speed Rail System (HSR) system, which is not completed yet, is integrated with the High-Capacity Rail System and consists of two main axes: one connecting Torino to Salerno, from the North to South via Milano, Bologna, Firenze, Roma and Napoli; while the other is connecting Torino to Venezia, from North West to North East via Milano (Fig. 2). Once completed, the HSR network will cover most of the Italian region, including the Napoli-Foggia link, and will connect the national system to the European one through four main corridors: the Baltic-Adriatic corridor (corridor 1); the Mediterranean corridor (corridor 3); the Helsinki-Valletta corridor (corridor 5); and finally, the Genova-Rotterdam corridor (corridor 6).

Table 2 summarizes the main features of the Italian HSR stretches.

It is worth noting that in Italy about the 16% of the total length of the national HRS system is located underground, within bored tunnels. This corresponds to a total of about sixty tunnels (including all tunnels with a length higher than 500 m). Overall, the HS tunnels length is very variable, with maximum lengths in the order of 15–20 km. For instance,

Table 2
Italian High Speed working routes features (extrapolated from http://www.rfi.it/rfi.html).

HS routes	*Length of rutes*	*Length of bored tunnel*		
-	*(Km)*	*(Km)*	*(% |single rute)*	*(% |total length)*
Torino - Milano	125	0	0	0
Milano - Brescia	66.6	0	0	0
Padova - Venezia	25	0	0	0
Milano - Bologna	182	0	0	0
Bologna - Firenze	78.5	73.80	94	7.65
Firenze - Roma	254	50.80	20	5.26
Roma - Napoli	205	25.75	13	2.67
Napoli – Salerno	29	3.50	12	0.36
Total Length	*965*	*153.85*	*–*	*15.94*

the Bologna-Firenze route counts several long HS tunnels (e.g., Vaglia tunnel 18713 m, Firenzuola tunnel 15285 m, Raticosa tunnel 10450 m, Pianoro tunnel 10481 m, Ginori tunnel 9259 m, Monte Bibele tunnel 9243 m, Sadurano tunnel 3855 m, Scheggianico tunnel 3558 m, Borgo Rinzelli tunnel 717 m, Morticine tunnel 654 m). The geo-localization of the tunnels of the HSR Italian network can be found at the National Geo-Portal (http://www.pcn.minambiente.it/viewer/), which provides a Web Map Service, WMS, implemented according to the standards ISO 19128, and a Catalog Service for the Web, supported by metadata.

Furthermore, it is important to consider that in the near future, the number of HS tunnels is expected to further increase considering the stretches under design and construction. The highest number of tunnels is along the Bologna-Salerno link in Central Italy, which is currently the only part in Italy belonging to the Helsinki-Valletta HS corridor. This stretch crosses a wide variety of geological contexts, and it includes different tunnel covers (e.g., ranging from few tens of meters in urban areas to hundreds of meters in mountain areas). For this reason, the

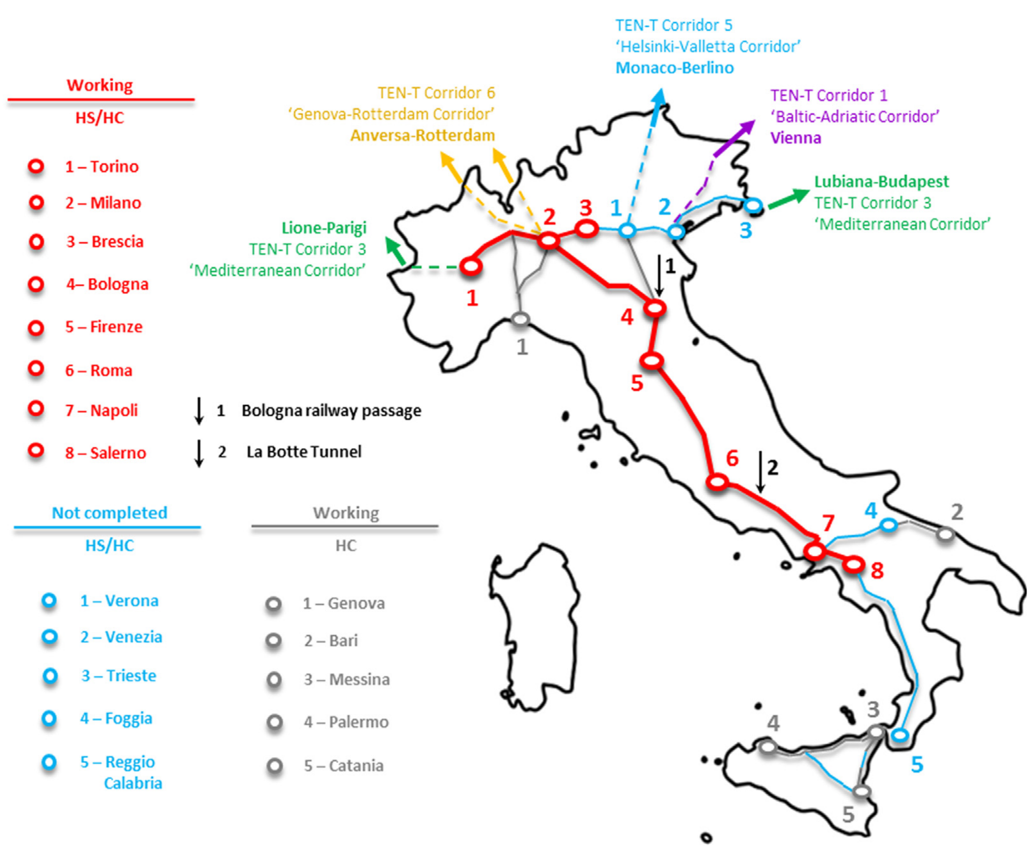

Fig. 2. Italian high speed/high capacity rail network.

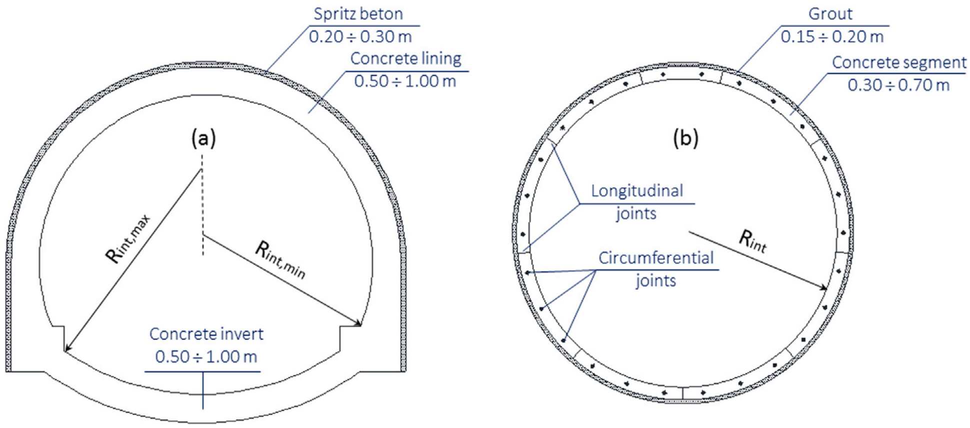

Fig. 3. Tunnel sections typology of Italian high speed rail network excavated with (a) traditional and (b) mechanized method.

tunnels are of different typology (i.e. lining technology, tunnel diameter, lining thickness, materials). Most of these tunnels (more of 80%) have been excavated with traditional methods, with the exception of some more recent cases, such as the Bologna railway passage and the Ginori tunnels that serves the Vaglia tunnel along the Bologna-Firenze stretch, which have been constructed with mechanized excavation. The latter technique has been increasingly adopted worldwide in the last ten years for rail and metro networks and it will be adopted for the excavation of new Italian HSR tunnel stretches.

Fig. 3 shows, as an example, the typical cross section of Italian HSR tunnels excavated with traditional (Fig. 3a) and mechanized methods (Fig. 3b). In the former case the final lining is continuous and cast-in-place and a polycentric cross-section is adopted to host two tracks, with a minimum value of $R_{int,min}$ (see Fig. 3a) of about 6 m, lining thickness variable between 0.50 m and 1.00 m, or more when necessary depending mainly on the tunnel depth and on the ground conditions; in the latter case a segmented and pre-cast concrete lining is installed and its section is circular, with a lower thickness compared with the former, generally variable between 0.30 m and 0.70 m, and the radius of the tunnel section tends to be lower as each tunnel host a single track.

Besides complex and variable ground conditions, the Bologna-Salerno stretch of the Italian HSR rail network crosses a seismogenic region characterized by one of the highest level of seismic hazard in the Mediterranean area. The Italian hazard map (Fig. 1) shows that along most of the Apennines the maximum horizontal ground acceleration expected at rock sites (a_g) with a probability of exceedance (P_r) equal to 10% in 50 years is 0.3 g. Furthermore, The Central Apennines have been recently struck by a devastating and long-lasting earthquake sequence between August 2016 and January 2017, which generated diffuse damage, estimated around the 1.4% of Italy GDP. The seismic sequence has been characterized by three large magnitude events (causing overall about 300 causalities): the M_w 6.0 Amatrice earthquake, occurred on August 24th, 2016; the M_w 5.9 Visso earthquake, on October 10th, 2016; and the M_w 6.5 Norcia earthquake, which occurred on October 30th, 2016 [33] with epicentre located in the vicinity of Norcia in the area of the Sibillini mountains. In occasion of these earthquakes, the largest values of Peak Ground Acceleration (PGA) have been recorded at short epicentral distances (< 15 km) at the stations Campi (CMI, 0.65 g), Norcia (NRC, 0.38 g) and Arquata del Tronto (RQT, 0.46 g) [34].

This study focuses, as examples to present the proposed approach, on two sites: a tunnel along Bologna-Firenze link, Case 1, and la Botte tunnel along Roma-Napoli link (in Ceccano, near Frosinone), Case 2. These target tunnels have been chosen for their position with respect to the most seismic zones of the region, and because they are representative of two different tunnel lining technologies. Both the considered areas show an important historical seismicity, as shown in Fig. 4(i.e., data from the Parametric Catalog of Italian Earthquakes by Rovida et al. [35] made available by the INGV, the Italian Institute of Geophysics and Volcanology; https://emidius.mi.ingv.it/CPTI15-DBMI15/description_CPTI15.htm).

The *Bologna railway passage (Case 1)* belongs to the Bologna-Firenze stretch and represents one of the most important interchange node of the national HS network. The 94% of Bologna-Firenze line is excavated in bored tunnels crossing the Apennine chain, characterized by relatively steep mountainous reliefs, with very variable tunnel overburden up to about 600 m in correspondence of the Raticosa tunnel on the Tuscan side, where the geology is characterized by sedimentary rock formations (i.e., marly-sandy formations and clayey flysch [36]).

Bologna railway passage is a very recent urban crossing long about 10 km, 6 km of which are in underground for the southern access to the HS station of the city. The latter consists of two parallel shallow tunnels excavated with an Earth Balance Pressure, EPB, Tunnel Boring Machine, TBM, of a diameter of 9.40 m. This technique was adopted to minimize interaction effects with the above structures of the urban area. The tunnel linings are segmented like in Fig. (3b), with 6 pre-cast concrete segments plus the key with a thickness of 0.40 m, while the internal diameter of the tunnels is equal to 8.30 m. The tunnels were excavated through two main formations, one alluvial deposit of Savena River with deposits of clay, and one of sandy soil. A detailed description of the site can be found in Do et al. [37].

La Botte tunnel (Case 2) belongs to the Roma-Napoli stretch that develops into a geological context mainly characterized by volcanic (pyroclastic soil, tuff and lava) and sedimentary rocks (i.e., flyshes, marly-sandy formations, limestones). Along this stretch, the tunnels cover is very variable, but it does not exceed 110 m of maximum height [36,38]. This structure is 1.5 km long bored tunnel, excavated with traditional method and variable cover along the stretch (maximum cover equal to 44 m, minimum cover equal to 20 m). Depending on the tunnel axis depth and the crossed ground conditions, the tunnel has a variable structural section (both the invert and the crown have a thickness variable between 0.60 m and 1.00 m, the area of excavation is variable between 125.3 m^2 and 151.9 m^2). The tunnel crosses two different predominant lithotypes: volcanic and sedimentary ones. Due to variability of the tunnel geometry and properties, the most seismically vulnerable section has been considered (i.e., lining thickness equal to 0.60 m and cover equal to 20 m, in sedimentary ground).

3.2. Method

Using the fragility curves for different levels of damage states (DSs), from the PGA value recorded at a target sites (e.g. a tunnel) the probability of damage (P_f) can be estimated. Fig. 5 shows a schematic application of the fragility curves for the estimation of P_f.

Whenever a dataset of past earthquake waveforms for a site of interest is not available, one simplest approach to assess the P_f that a

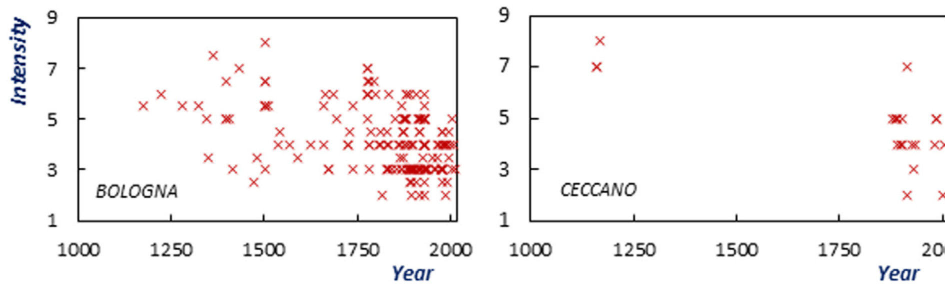

Fig. 4. Historical seismicity of Bologna - CASE 1 - and Ceccano - CASE 2 - (Istituto Nazionale di Geofisica e Vulcanologia, [35]).

structure can experience during its lifetime is to exploit the Probabilistic Seismic Hazard Analysis (PSHA) for extracting the ground motion level (i.e., the PGA for instance) that can occur at the structure site within a given reference period and different exceedance probabilities (e.g. P_r = 81%, P_r = 63%, P_r = 10%, P_r = 5%, as proposed in the Italian code). This approach was applied to both tunnels to calculate a "background" level of P_f, as detailed in the following section (§3.3.1).

While such a "background" P_f can be obtained straightforward from the PSHA, it is worth noting that this procedure does not provide any direct indications about the location of the seismic sources that contributed to the hazard. By contrast, during the design of EEWS, this is a key piece of information, being the location of seismic sources necessary to compute the lead-time, which in turn is needed to understand the effectiveness of the early-warning. A possible strategy to overcome this issue is to exploit the disaggregation of seismic hazard, as schematically shown in Fig. 6, which provides insights into the earthquake scenarios driving the hazard at a given ground motion level [39] as function of the selected intensity measure and the mean return period. The proposed procedure (§3.3.2) makes use of the disaggregation analysis applied to the PSHA defined for the Italian territory in terms of peak ground horizontal acceleration, PGA, to identify 'virtual sources' (i.e., in terms of epicentral distance and magnitude) that provide the higher contribution to the hazard at a target location. Disaggregation maps are, indeed, expressed in terms of magnitude (M), source-to-site distance (R_{epi}) and the contribution (w%) to the hazard. In other words, the disaggregation analysis permitted to identify seismic events that from the hazard perspective are of interest for the structures at hand. Then, for each pair of (M-R_{epi}) derived from the disaggregation analysis, the PGA at the target site where the tunnel is located is computed by a ground motion prediction equation (GMPE) (Fig. 6); in this study the GMPE proposed by Bindi et al. [40] for Italy has been adopted.

The set of PGA values obtained for the target site represents the input matrix for the fragility curves. By means of them, for each tunnel a matrix of probability of damage, P_f, is defined for each damage state. Furthermore, the set of virtual seismic sources are used also to estimate a lead-time matrix, which allows assessing the time available to implement protective actions at the target structure to be protected (and therefore it allows to select which EEW approach between on-site and regional is most indicated). To this purpose, for each pair of (M-R_{epi}) the lead-time is computed assuming: (i) P-wave and S-wave velocities of 5.5 km/sec and 3.0 km/s, respectively; (ii) a P-waves time window length of 1 s for estimating the magnitude until M = 7 [41,42]; (iii) the time spent for computation and telemetry together equal to 1 s, assumed on the basis of the experience with the EEW and management system PRESTo operating in Southern Italy [43]; (iv) for each site, according to the seismic zone of the seismic hazard analysis where it is placed, a hypocentral depth is assumed equal to the "efficient depth", which is defined as the depth interval where 90% of the events occur.

For the two test-cases considered in this study, the lead-time and acceleration matrices have been computed. In turn, considering all M-R_{epi} pairs, the PGA maps have been used in combination with the fragility curves to produce the final matrix with the probability of minor, moderate and extensive damage.

3.3. Results

3.3.1. "Background" damage probability, P_f

The PSHA has been used to evaluate the expected seismic scenario for the selected cases of study. Hence, as a function of the coordinates of the considered target sites, the PGA values in correspondence of the 50° percentile for each selected probability of exceedance (P_r = 81%, P_r = 63%, P_r = 10%, P_r = 5%; points yellow, blue, green and red in Fig. 7, respectively), have been extracted from the Italian seismic hazard database through the Interactive Seismic Hazard Maps made available by the INGV (http://zonesismiche.mi.ingv.it). Then, these values have been multiplied by the amplification site coefficient (S_s), as indicated by the Italian code NTC 2008, to calculate the PGA at the surface as function of the ground type, assuming a reference period V_R for the infrastructures equal to 50 years.

The derived PGAs represent the input for the fragility curves and allow assessing the damage probabilities associated to each structure. According to the tunnel sections typology, the numerical fragility

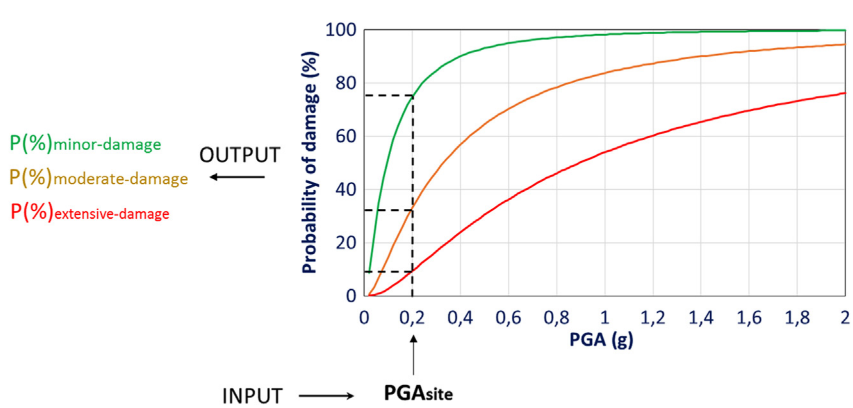

Fig. 5. Schematic application of the fragility curves.

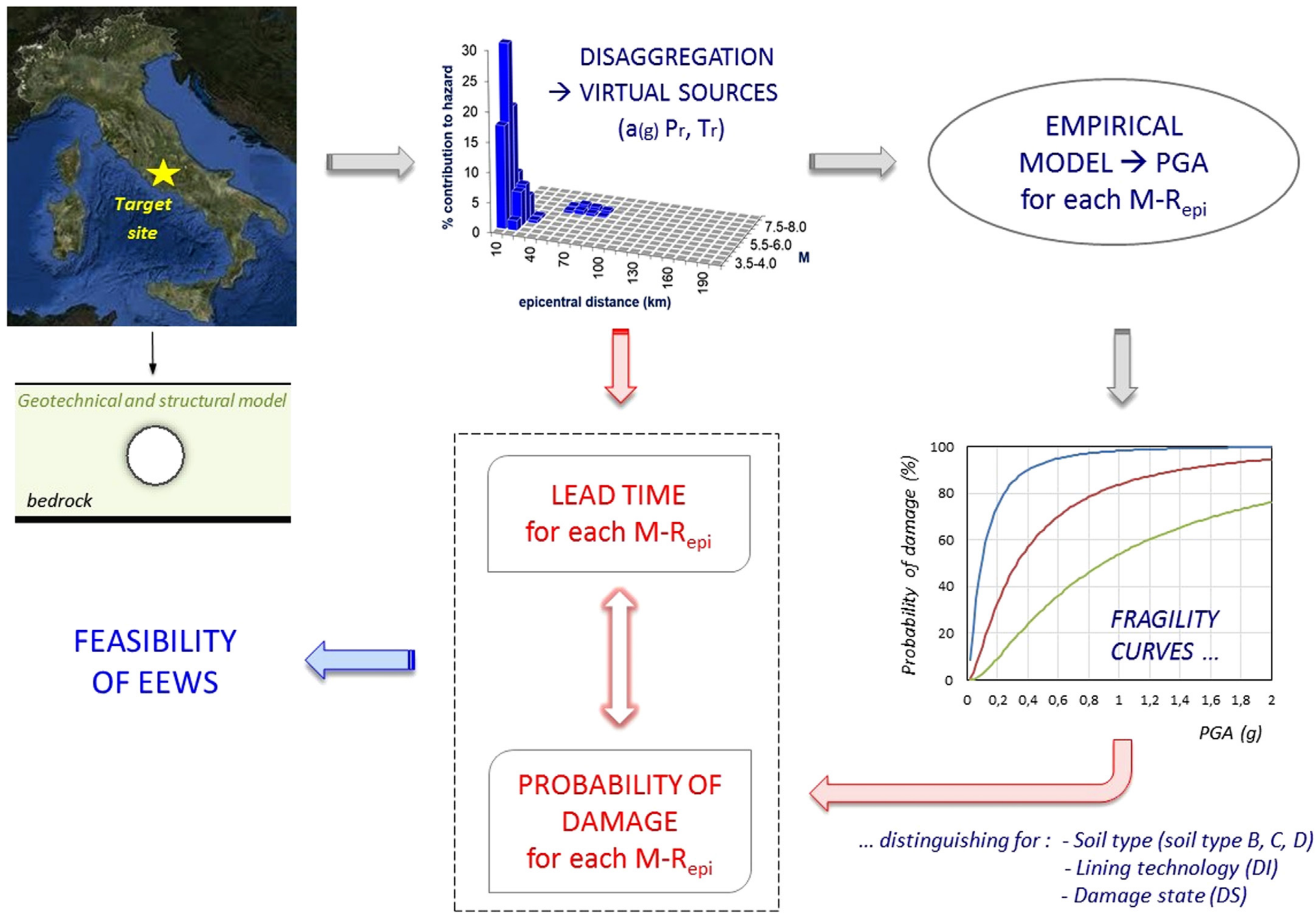

Fig. 6. Layout of the proposed method based on the disaggregation of the PHSA to evaluate the probability of seismic damage of underground tunnels and the related feasibility of EEWS.

curves estimated by Fabozzi et al. [19] and that from Argyroudis and Pitilakis [15] have been used for the 'Bologna railway passage' (CASE 1) and 'la Botte tunnel' (CASE 2), respectively.

Tables 3, 4 present the probability of minor, moderate and excessive damage, P_f, expected for the target tunnels for different probability of exceedance, P_r. We observe that P_f decreases when passing from minor to extensive damage state level. Moreover, it increases by reducing the probability of exceedance P_r, because this corresponds to an increase of the maximum surface acceleration expected at the target site.

Comparing the results obtained for both cases, Tables 3, 4 show that in CASE 1 the probability of having damages is higher than in CASE 2, at each damage level, although the maximum acceleration expected for the former case is slightly lower than those of the latter. This result is likely related to the fact that in the segmental lining (CASE 1) the structure is highly prone to localized rotations at the joints, which may induce damage, while in the case of the continuous lining (CASE 2) the vulnerability is related to the arising of cracks in the concrete lining, which generally requires a larger seismic demand. Interestingly, the

Table 3
Probability of minor, moderate and extensive damage (%) calculated in function of Pr in the target sites of Bologna railway passage.

CASE 1		*Probability of Damage, P_f, %*		
Pr (%)	*a_g*S_s*	*Minor*	*Moderate*	*Extensive*
81	0.064	34.65	6.66	0.85
63	0.080	44.40	10.61	1.65
10	0.200	74.79	33.06	9.31
5	0.256	80.61	40.44	13.03

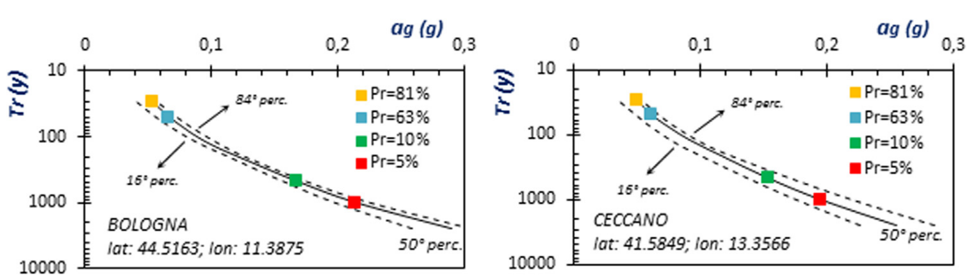

Fig. 7. PGA values for different mean return periods (T_R = 30 y, T_R = 50 y T_R = 475 y T_R = 975 y) and corresponding probability of exceedance (Pr = 81%, Pr = 63%, Pr = 10%, Pr = 5%) calculated for the target tunnels.

Table 4
Probability of minor, moderate and extensive damage (%) calculated in function of Pr in the target sites of La Botte tunnel (along Roma-Napoli link).

CASE 2		*Probability of Damage, P_f, %*		
Pr (%)	*a_g*S_s*	*Minor*	*Moderate*	*Extensive*
81	0.074	0.20	0.03	0.01
63	0.092	0.70	0.13	0.04
10	0.229	10.70	3.48	1.51
5	0.288	18.00	6.89	3.31

expected probability of damage in correspondence of the extensive damage level is very low in both cases; a result confirmed by the fact that in Italy there have not been cases of tunnel structural collapse. On the contrary, the probability of damage for minor and moderate damage levels is higher. For the CASE 1, the probability of damage starts from values of 35% and 7% for minor and moderate damage level, respectively, in correspondence of P_r equal to 81%. Then, it increases up to values of 80% (minor damage level) and 40% (moderate damage level) in correspondence of the P_r equal to 5% (Limit State of Collapse, according to the Italian Building Code). For the CASE 2, instead, P_f is almost null in correspondence of P_r equal to 81% and 63% (Serviceability Limit States). However, it becomes not negligible for P_r equal to 10% and 5% (Ultimate Limit States), for which P_f is equal to 20% (minor damage level) and 7% (moderate damage level) in correspondence of P_r equal to 5% (Limit State of Collapse).

Despite the analysed cases can be considered well representative for a large number of existing (or under completion/design) Italian HSR tunnels that share similar structural, geotechnical and seismic characteristics, it is worth noting that different seismic scenarios (i.e., higher values of PGA) are expected for other Italian HSR tunnels as a function of site. This is rather obvious projecting the HSR route on the Italian seismic hazard map (Fig. 1) and considering the spatial variability of ground motion during the last seismic events of the Central Italy as well. Whereby, parametric analyses of tunnel damage probability have been carried out in the case of continuous and segmented tunnel lining and for different values of PGA aiming at covering different possible scenarios that can be well representative for existing and future HSR Italian tunnels. Table (5) shows an assessment of the "background" damage expected for Italian HSR tunnels, for different lining structures.

3.3.2. Loss-driven earthquake early warning and rapid response feasibility

In this section, a procedure to assess the probability of damage P_f based on a disaggregation of PHSA is presented. This is applied to a feasibility analysis for a loss-drive EEW and rapid response system for the tunnels of CASE 1 and CASE 2.

Table 5
Parametric analyses to calculate the probability of minor, moderate and extensive damage (%) for continuous and segmental lining technology in function of different values of a_g.

Probability of Damage, P_f, %			
a_g	*Minor Continuous – Segmental tunnel lining*	*Moderate Continuous – Segmental tunnel lining*	*Extensive Continuous – Segmental tunnel lining*
0.20	12.73–74.80	5.57–33.06	2.88–9.31
0.25	20.00–80.62	9.79–40.44	5.49–13.03
0.30	27.47–84.75	14.65–46.80	8.74–16.75
0.35	34.70–87.72	19.88–52.20	12.48–20.38
0.40	41.47–89.97	25.21–56.89	16.52–23.88
0.50	44.58–93.54	23.98–64.23	14.46–30.94
0.60	54.94–94.32	32.77–70.75	21.20–36.47
0.70	63.47–96.18	41.06–74.31	28.12–41.11

Figs. (8a) and (9a) show the results of the disaggregation analysis through which indications about the location and magnitude of the seismic sources that mostly contribute to the hazard are derived.

Taking into exam CASE 1 for P_r = 63%, the disaggregation map in Fig. 8a shows that the seismic threat responsible for the highest contribution to hazard is located at a distance between 10 and 20 km from the target site with a magnitude between 4.5 and 5. The peak ground acceleration (PGA) at the tunnel related to this scenario is equal to 0.04 g (Fig. 8b), which corresponds to a probability of minor, moderate and extensive damage equal to 20%, 0%, and 0%, respectively (Fig. 8d, e, f). The correspondent lead-time instead, is equal to 1.6 s (Fig. 8c). It is worth noting, however, that the maximum PGA expected at the target is about 0.2 g, to which corresponds a probability of minor, moderate and extensive damage equal to about 70%, about 35% and lower than 10%, respectively. This scenario corresponds to a source located at a distance between 5 km and 10 km from the site and a magnitude range from M = 6 to M = 6.5. Despite this scenario would be very important in terms of ground motion at the target site and damage probabilities, it is worth to consider that it is associated to a very low contribution to the hazard, and therefore it is very unlikely to occur. These two cases represent the end members (i.e. the one with the highest occurrence probability and the worst one in terms of PGA) of a wide number of possible scenarios. From this point of view, this approach is a very versatile tool, which allows to investigate different scenarios in terms of PGA and probability of occurrence.

Considering the lead-time of 1.6 s for the seismic threats with highest contribution to the hazard (R_{epi} = 10–20 km, M from 4.5 to 5), our feasibility analysis suggests for CASE 1 a threshold–based on-site EEWS system (i.e., a system made of 2–3 accelerometric stations installed nearby the tunnel).

It is worth noting that our definition of lead-time provides conservative values, being it based on the theoretical arrival time of the S-wave at the target site. Clearly other approaches could be followed, as for instance, the one proposed by Festa et al. [47], where the lead-time is measured on waveforms as the difference between the instant at which the ground velocity overcomes for the first time the threshold value of 3.4 cm/s (i.e., the lower boundary of the VII degree in the instrumental intensity scale proposed by Faenza and Michelini [48]) and the P-wave arrival. In this last case, the definition of the lead-time is related to the effective arrival of the ground shaking of interest for an EEWS and it would provide for the targets selected in this work larger lead-times.

A similar analysis has been carried out also for CASE 2. Fig. (9a) shows that the seismic threat responsible for the highest contribution to hazard is located in this case at about 5 km from the target site, but again it is associated to a low magnitude (i.e., from M = 4 to M = 4.5). The correspondent PGA is equal to 0.05 g (Fig. 9b), which would determine a probability of damage almost nil for minor, moderate and extensive damage state (Fig. 9d, e, f). The correspondent available lead-time is equal to 0.6 s (Fig. 9c). The maximum PGA expected at the target is 0.22 g and corresponds to a source located at about 15 km of distance from the site, magnitude in the range M = 7 to M = 7.5, but, like in the previous case, a very low contribution to the hazard. This threat is associated to a probability of minor, moderate and extensive damage equal to about 5%, about 4% and lower than 2%, respectively (Fig. 9d, e, f).

Hence, for CASE 2 the seismic virtual threat to be considered more likely, that is the first mentioned threat with the higher contribution to the hazard (R_{epi} = 0–10 km, M = 4–4.5) would produce negligible damage. It should be noted that the low probabilities of damage associated to CASE 2 are due to the lower vulnerability of the continuous lining with respect to the segmental lining of CASE 1, of course for the same PGA level. As for CASE 1, also for CASE 2 the low lead-time associated to the seismic threat with the highest contribution to the hazard suggests the adoption of a threshold–based on-site EEWS.

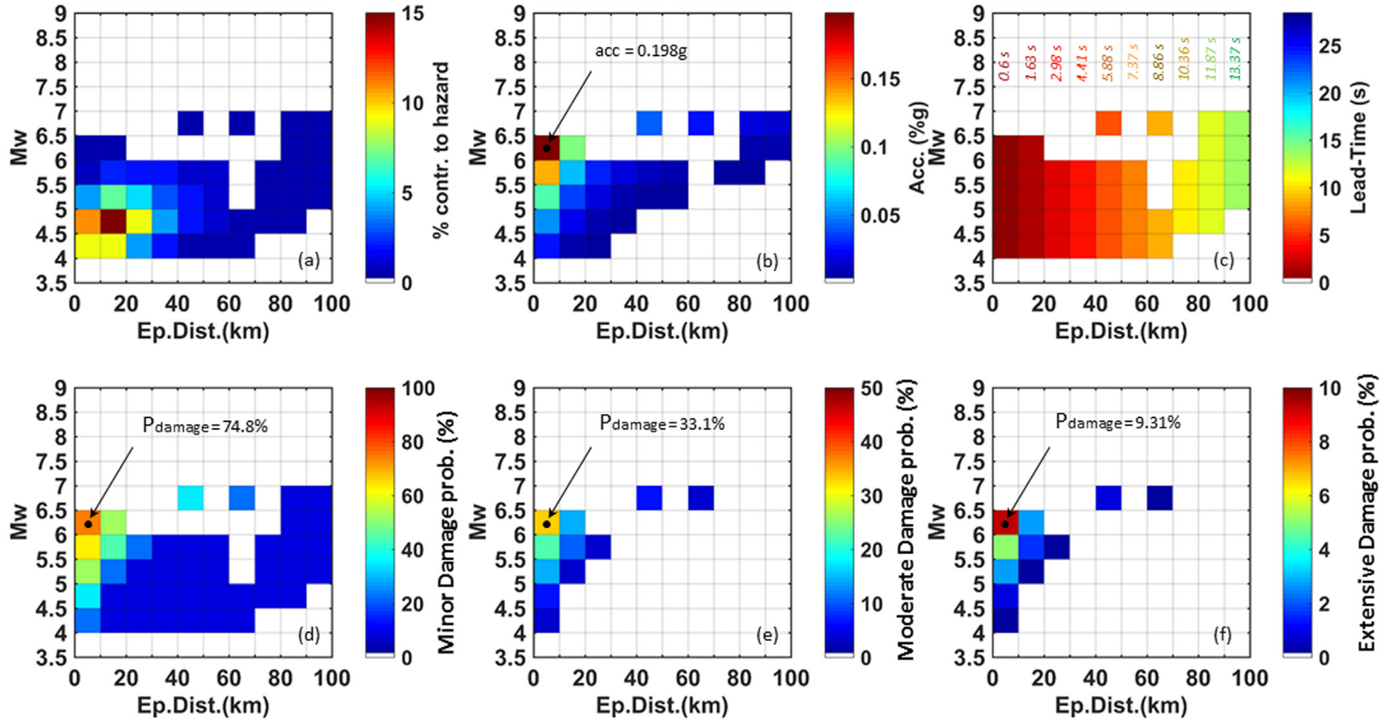

Fig. 8. (a) Disaggregation (INGV, http://zonesismiche.mi.ingv.it), (b) acceleration and (c) lead-time maps of target site of Bologna HS railway passage and maps of probability of damage for (d) minor, (e) moderate and (f) extensive damage level (Pr = 63%).

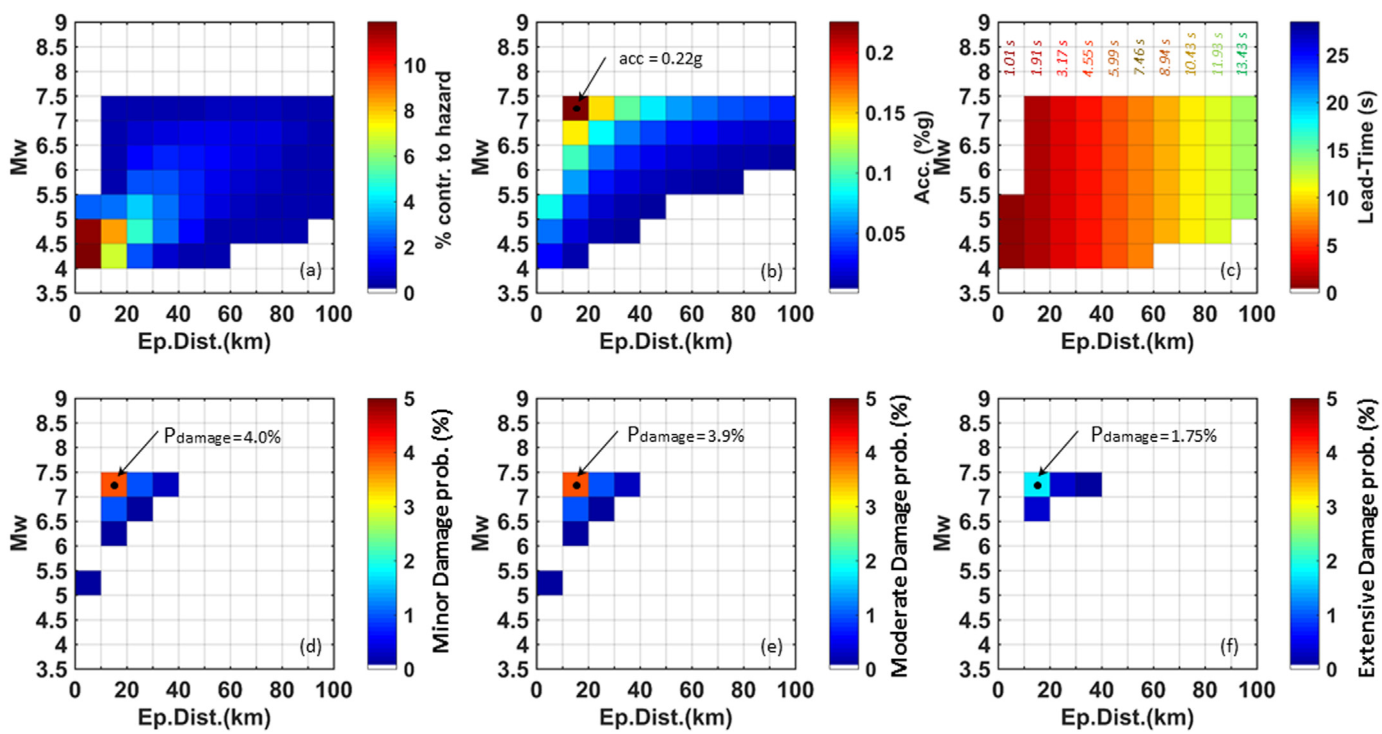

Fig. 9. (a) Disaggregation (INGV, http://zonesismiche.mi.ingv.it), (b) acceleration and (c) lead-time maps of target site of La Botte HS railway tunnel and maps of probability of damage for (d) minor, (e) moderate and (f) extensive damage level (Pr = 63%).

4. Discussion and conclusions

The seismic vulnerability of underground structures is an issue of concern for modern societies in relation to the strategic importance that such structures have. The probability of damage, P_f, even if not as high as in the case of aboveground structures, could lead to high levels of seismic risk associated to the serviceability and functionality of the tunnel structure, with consequent economic impact on their management. Due to the lack of specific national and international guidelines for the seismic design of tunnels and underground structures, such a topic has received an increasing attention in the last years.

This works moves from this context, and it focused on assessing the feasibility of a loss earthquake early warning systems based on the real-time estimation of the expected damage probability and lead-time, in particular for the Italian High Speed Rail system.

Taking into consideration two typology of tunnels belonging to the Italian HS rail network, a “background” damage probability has been calculated using the Probabilistic Seismic Hazard Analysis (PSHA). The analysis could be easily adopted at the early stages of design of new tunnels. Moreover, since the approach enables rapid responses, it can be seen as a tool to increase the rail network resilience after a moderate to strong earthquake.

Afterwards, a further procedure for computing P_f at a target sites has been presented, where a direct indication about the location of the ‘virtual seismic sources’ and their contribution to the seismic hazard (i.e. M-R_{epi} derived from the disaggregation analysis) are considered. The importance of considering these pieces of information (i.e., M-R_{epi}) lies in the fact that they allow to assess P_f considering a specific hazard scenario (spatially constrained and probabilistically defined in time). Therefore, defining scenarios of interest for a target allows to evaluate the lead-time (i.e. the time available before the arrival of S-waves to the target), which in turn supports the design of the most effective seismic risk protection action to be implemented in real-time at the specific target.

The procedure proposed in this work can be used at different stages of the design of new tunnels. Moreover, it shows potential also for the implementation of Earthquake Early Warning Systems on existing tunnels. In this last case, the proposed procedure can be adopted to evaluate the feasibility of EEWS and to schedule the ordinary actions to monitor, manage and maintain the tunnel structures.

It must be considered that Italy has a high potential for a nation-wide EEWS, given the more than 750 accelerometric stations installed across the whole Country's active seismic zones, despite the need for communications network and data streaming to be upgraded. A significant number of these stations (i.e., ~500) are part of the Italian accelerometric network (http://www.protezionecivile.gov.it/jcms/it/ran.wp) managed by the Italian Department of Civil Protection (Dipartimento della Protezione Civile, DPC), whose data are mainly used for emergency response services [44]. Picozzi et al. [45] already have explored the scientific feasibility of a nation-wide EEWS in Italy that would exploit the RAN and the EEWS software PRESTo [43]. The results found by Picozzi et al. [45] suggest that a nation-wide RAN-PRESTo EEWS might provide reliable alert messages within 5–10 s after the occurrence of a moderate to large earthquake. Maximum lead-times of about 25 s were inferred for earthquakes generated in the higher seismic hazard zones. The present work is therefore well-integrated in a series of studies that are exploring the EEW feasibility for different targets/end-users (e.g., Picozzi et al. [46] and Emolo et al. [47] considered the public schools in each municipality of the Campania region in southern Italy, Festa et al. [48] following the 2016–2017 Central Italy seismic sequence) for the future application in Italy of systems capable of the real-time seismic risk mitigation.

An important issue of concern of the proposed method is represented by the fragility curves which express the seismic vulnerability of the structure for different hazard levels, associated to a specific failure mechanism. It seems worth noticing that in the two cases presented in the paper the tunnel depth and the ground conditions comply with the assumption at the base of the adopted fragility curves. However, in order to extend the range of applicability of the proposed procedure, further analytical fragility curves should be developed to cover the whole range of existing tunnels. In fact, as it is emerged from the literature review (§2) the analytical fragility curves available for underground tunnel structure are not very numerous [14–19] and sufficient to cover all the possible cases since several variables need to be considered in a soil-tunnel interaction problem: tunnel depth, lining thickness, stiffness of the materials, bedrock position, particular seismic ground conditions and the presence of joints in segmental lining. Furthermore, the above-mentioned fragility curves have been derived considering in plane strain conditions through pseudo static or full dynamic analysis, and only in the latter the influence of the non-linear and irreversible behaviour of soil was considered.

In addition, the analytical fragility curves adopted for existing tunnels should include also the effect of aging of the lining which likely leads the probability of damage to increase up to a value of 50% in some ground conditions and life cycle of the structure [17]. This last aspect, in particular, could have an effect in the analysed case of La Botte tunnel which was constructed in the second half of the last century.

In conclusion, the target tunnels analysed in this work can be considered representative of a certain number of existing Italian tunnels of HSR with similar structural characteristics and placed at sites with similar geotechnical and seismic properties. Different scenarios may be expected for other Italian tunnels due to the different location with respect to the seismic hazard and the different structural and geotechnical characteristics. Hence those tunnels could experience different seismic actions compared to the analysed cases. Concerning this point, given all the above discussed assumptions and approximations, the parametrical results shown in Table 5 may represent a rapid and useful tool for a rapid pre-assessment of the expected probability of tunnel damage as a function of their location.

Appendix A. Supporting information

Supplementary data associated with this article can be found in the online version at http://dx.doi.org/10.1016/j.soildyn.2018.05.019.

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