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ORIGINAL RESEARCH PAPER

A probabilistic strategy for seismic assessment and FRP retrofitting of existing bridges

R. Morbin · M. A. Zanini · C. Pellegrino · H. Zhang · C. Modena

Abstract This study proposes a holistic probabilistic framework to evaluate existing road and railway bridges after an earthquake by means of analytical fragility curves and visual inspections. Although visual inspections are affected by uncertainties, they are usually considered in a deterministic way, while in this work they are taken into account in a probabilistic point manner. Moreover, extra focus is given on retrofitting interventions by means of Fiber Reinforced Polymer materials and their costs. A probabilistic methodology is formed to evaluate possible standardized interventions on existing bridges after a seismic event. The proposed framework, consists of six basic steps and it is applied on a reinforced concrete bridge case study, which is a common structural typology in Italian roadway infrastructural networks. The main aim is to provide useful information to public authorities in order to decide whether or not they should allow traffic over the bridge and whether to repair immediately earthquake-damaged bridges. The outcomes of this framework can be used to improve procedures used for the seismic assessment of the whole road and railway networks to better plan emergency, post-emergency actions and define a general priority for an optimal budget allocation.

1 Introduction

The fast socio-economic development of many urban areas has often been characterized by the construction of new infrastructures to meet the increasing transportation demands. Transport

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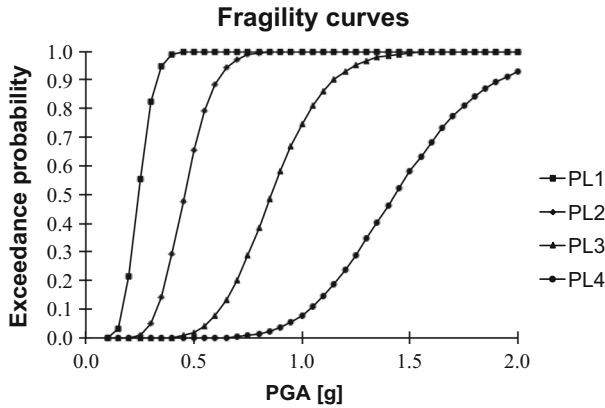
Fragility curves

Fig. 1 Example of fragility curves for existing bridges considering four performance levels (PLs)

networks are indeed essential for various economic and strategic activities immediately after a catastrophic event. From the analysis of seismic events that worldwide occurred in the last decades, it has been observed how the most vulnerable elements in transport networks are mainly bridges but also tunnels, retaining walls, etc, the damage of which may seriously affect transport mobility. These considerations can be easily understood analysing many areas of the Italian national territory where often the connection between urbanized centres is provided by a few links of the network and, for serious damages of bridges and viaducts, it easily runs the risk of damaging all these few links causing the isolation of those urbanized centres. It is also known that the Italian territory, as other regions in the Mediterranean area, is characterized by high seismic risk, and seismic events involve significant negative economic consequences, due to the high population density in many areas of these countries.

Furthermore, bridges can experience structural problems due to environmental conditions and natural disasters: concrete cover damage that exposes bars to atmosphere, steel corrosion, concrete damage by icing cycles, ageing of structural materials are some causes leading to the degradation of reinforced concrete bridges' mechanical properties (Zanini et al. 2013; Pellegrino et al. 2014). The allocation of limited budget resources for the retrofit is a key issue in stock management: the optimal distribution of a limited budget is a challenge connected to prioritization issues in order to maximize the service level of an infrastructural system.

Since the importance of this topic is undoubtable, this study is focused on the seismic vulnerability assessment of existing bridges by means of the formulation of a new probabilistic framework based on visual inspections on structures and analytical fragility curves. Such a framework is widely considered as one of the most performing tools in order to assess existing bridges' seismic vulnerability (Shinozuka et al. 2000a; Monti and Nisticò 2002; Franchin et al. 2006; Padgett and DesRoches 2008; Zanini et al. 2013; Carturan et al. 2014). Seismic fragility curves (Fig. 1) are relationships describing the probability of a structure being damaged beyond a specific damage state, or performance level (PL), for various levels of a specific ground shaking intensity measure (IM), typically peak ground acceleration (PGA) or spectral acceleration at 1s ($S_a[1s]$) values.

Various retrofitting techniques have been used to reduce seismic vulnerability of existing bridges. In this context, particular interest is dedicated to the confinement of piers with innovative materials, such as fiber reinforced polymer (FRP) composites (Teng et al. 2002; Tastani et al. 2006; Pellegrino and Modena 2010). The impact of these FRP interventions is

evaluated under a probabilistic point of view in terms of its influence on the bridge fragility curves.

The whole procedure is applied step by step on a reinforced concrete (RC) bridge case study characterized by a common structural typology in Italy (multi-span, simply supported girder bridges) to highlight its usefulness in bridge management activities. The results of the proposed framework can be used to better plan emergency and post-emergency responses and prioritizing resources to achieve optimal budget allocation.

2 The probabilistic framework

Society depends on the transportation infrastructure for economic, environmental, life-quality, safety and employment protection reasons. Therefore, the failure even of a single bridge can cause various severe problems. Considering the number of structures reaching a critical age, innovative test and assessment tools as well as methods are required in order to avoid an infrastructure breakdown. The proposed probabilistic framework is made up by six steps: it includes seismic vulnerability assessment of existing bridges, in particular after a quake main shock, and evaluations on possible retrofit interventions on damaged bridges. The main aim is to provide useful information to public authorities to decide whether or not allowing traffic over a bridge after a seismic event and whether or not to repair earthquake damaged bridges immediately, in order to manage an optimal budget allocation.

In the first two steps the aim is to collect information for the existing bridges in a transportation network (e.g., photographs, information about materials, static scheme, etc.) and deriving the respective fragility curves: the availability of main information about existing bridges is essential in emergency and post-emergency phases.

The other four steps are related to the planning of seismic emergency and post-emergency phases: a methodology to decide if non-destructive evaluation (NDE) methods on structures (e.g., visual inspections) are needed after a seismic event (step 3); the generation of fragility curves for damaged bridges considering NDE methods uncertainties (step 4); a methodology to decide whether or not allowing traffic over damaged bridges (step 5); and a methodology to decide whether or not repairing immediately earthquake damaged bridges (step 6) are described. Although the framework can be generally applied to every bridge typology, the focus is given on reinforced concrete (RC) existing multi-span simply supported girder bridges, which represent a common bridge structural scheme in Italy.

2.1 Step 1: Collecting bridges information in a database

A web database of the network road bridges with photographs, information about materials, static scheme, location and finite element models (FEMs) has to be created. Bridges' fragility curves and information about specific active seismogenic zones (SZ), have to be also embedded. This database is useful because, after a seismic event, the speed of finding information about characteristics and structural schemes of the examined bridges is of paramount importance. An example of this database type is the Italian BRidge Interactive Database (I.Br.I.D.) Project performed by the Department of Civil, Environmental and Architectural Engineering—University of Padova (VV.AA. 2006–2012). Figure 2 shows the structure of a I.Br.I.D. bridge folder webpage.

The Interactive Bridge Database provides information and data on bridge structures given by the members of the project as well as by the registered users, who are dealing with inspection and maintenance of such structures. The registered users can add new data to the
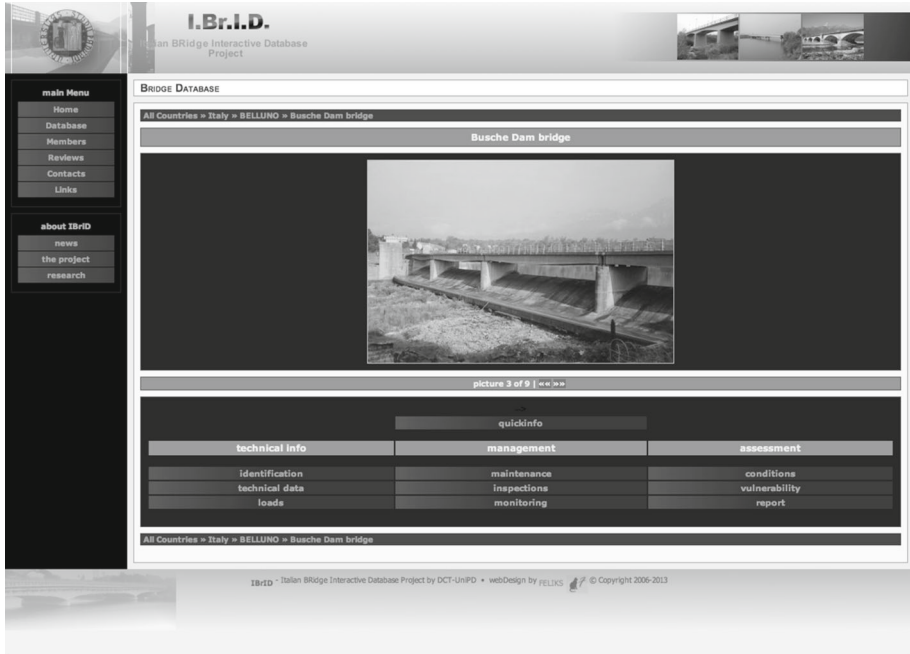
Fig. 2 Representation of an I.Br.I.D. bridge folder webpage (V.V.A.A 2006–2012)

database by filling in the requested forms and browsing through the contributions of the other users. The data are available free of charge for further Research and Development (R&D) activities. The database provides different types of information and details on the seismic vulnerability analysis are given in the section '*Research Issues*'.

The layout of the tables is designed to define a number of information objects. The objects are combined to deliver the contents of requested entities. An advantage with this design is that it enables defining new entities in the future without significant alteration of the existing data tables. The need for continuous maintenance of infrastructure systems, classification and updating of the information on structural conditions makes this database an important tool to reach these goals.

2.2 Step 2: Generation of fragility curves

Fragility curves need to be calculated for each bridge and integrated in the above-mentioned database. Fragility curves can be empirical or analytical. Empirical fragility curves are usually based on bridge damage data from past earthquakes, without considering any specific analysis of bridge. Various methods have been developed to generate empirical fragility curves: for example Shinozuka et al. (2000a) used observations of bridge damages in 1995 Kobe earthquake. Another method used in Europe is the RISK-UE method (Monge et al. 2003), based on HAZUS-99 (FEMA 2001) procedure.

Analytical fragility curves are developed through dynamic analyses of the structure and they can be used when a large amount of data from past earthquakes are not available. Most of the analytical methods of the literature consist of three steps: simulation of ground motions, modelling of bridges taking into account the uncertainties and generation of fragility
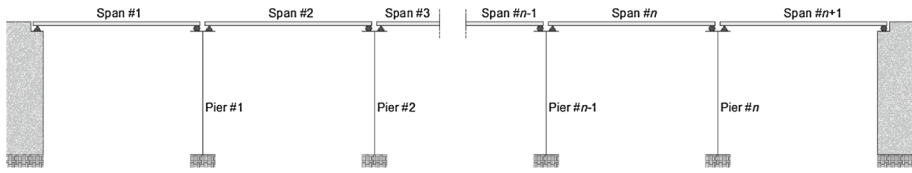
Fig. 3 Structural scheme of a bridge with $n+1$ simply-supported spans and n piers to which Eq. (4) is referred

curves from the seismic response data of the bridges. Seismic response can be obtained from different types of analysis: elastic spectral analysis (Hwang et al. 2000), non-linear static analysis (Shinozuka et al. 2000b), non-linear time history analysis (Choi et al. 2003; Morbin et al. 2010; Zanini et al. 2013).

A number of studies have showed that the lognormal distribution fits well average seismic demand on the structure S_d (Cornell et al. 2002; Nielson and DesRoches 2007):

$$S_d = e^A IM^B \tag{1}$$

this law can be represented by a straight line having the following equation (*e* is the Euler's constant):

$$ln\,(S_d) = \lambda = A + B\,ln\,(IM) \tag{2}$$

where IM is the seismic intensity measure, A and B are two coefficients calculated by linear regression of the entire data set, which depend on the probabilistic characterization of materials properties (Cornell et al. 2002), and λ the average value related to a specific IM. After finding A and B coefficients and the dispersion, the fragility curve becomes a cumulative lognormal distribution. The lognormal distribution f_d is expressed as:

$$f_d\,(d_{PL}|IM) = \frac{1}{\sqrt{2\pi}\,\varepsilon d_{PL}}exp\left[-\frac{1}{2}\left(\frac{ln\,d_{PL}-\lambda}{\varepsilon}\right)^2\right] \tag{3}$$

where d_{PL} is the considered damage (related to a specific PL) and ε is the dispersion calculated by Eq. (2).

The procedure is referred to a single pier ($P_{f,PL,pier}$). Considering a bridge with $n+1$ simply-supported spans and n piers (as shown in Fig. 3), the probability of the entire structure ($P_{f,PL,system}$) to achieve a certain PL for each IM is:

$$P_{f,PL,system}\,(IM) = 1 - \prod_{pier=1}^{n}\left[1 - P_{f,PL,pier}\,(IM)\right] \tag{4}$$

Since piers are considered the most vulnerable elements of the bridge (Shinozuka et al. 2000b), the PLs are defined in relation to the pier kinematic ductility:

$$d_{PL} = \frac{x_{max}}{x_y} \tag{5}$$

where x_{max} is the maximum horizontal displacement of a target point (e.g. the point at the top of the pier) during the time history of an earthquake and x_y is the horizontal displacement at the same point for steel yielding in the base cross-section of the pier. Four PLs are considered (Choi 2002):

*PL*1: $d_{PL} = 1$ light damage,
*PL*2: $d_{PL} = 2$ moderate damage,

Horizontal Target Spectrum and matched accelerogram
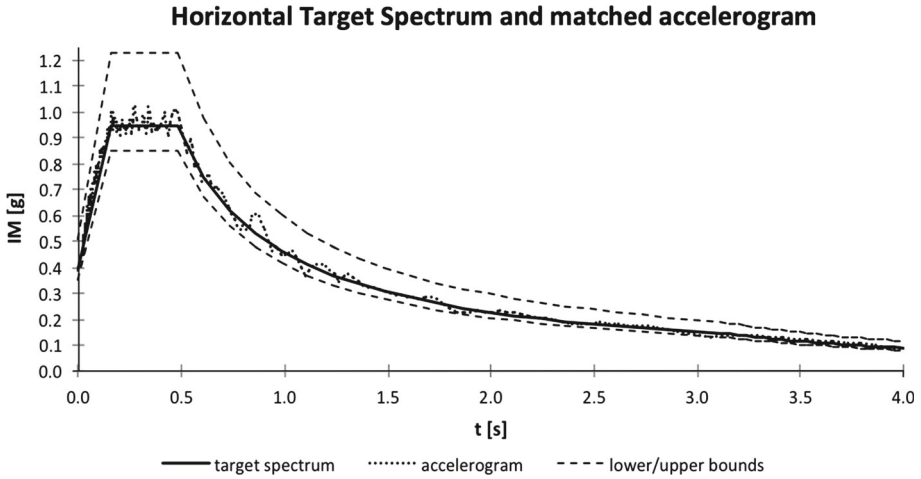
Fig. 4 Example of a horizontal target spectrum with a matched artificial accelerogram by Vanmarcke (1976) method

PL3: $d_{PL} = 4$ extensive damage,
PL4: $d_{PL} = 7$ complete damage.

Moreover, a shear failure PL can be defined, e.g., piers shear failure. This PL is unique because shear failure is considered a brittle failure. Hence, it could be defined as in Eq. (5) where x_y is replaced by x_s, the horizontal displacement of the target point in relation to the pier shear failure corresponding to the base cross-section of the pier, if the pier is considered as a cantilever.

The choice to consider flexural PLs, based on kinematic ductility, and/or shear failure PL depends on the pier behaviour subjected to horizontal forces: in terms of displacements and strength, it is influenced by sectional behaviour, geometrical characteristics, rotations at foundation level, presence of an isolation device, etc. This topic is reviewed in detail in Miranda et al. (2005).

Structural seismic response is obtained by non-linear dynamic analysis (NLDA) utilizing artificial accelerograms, obtained via the stochastic vibration method (Vanmarcke 1976): this method is implemented in SIMQKE code (Gasparini and Vanmarcke 1976), which calculates power spectral density function from a defined response spectrum and uses this function to derive the amplitudes of sinusoidal signals having random phase angles uniformly distributed between 0 and 2π. The sinusoidal motions are summed to generate independent accelerograms (compatible with the response spectrum). In this work the target spectra are horizontal and vertical elastic response spectra with 5 % damping coefficient and 4 s largest period. Artificial accelerograms total duration is 20 s: the stationary part of the accelerograms starts after 2 s and its duration is 10 s, according to Italian Ministry of Infrastructures (2008). The response spectra ordinates of these accelerograms are in the range of 90 % (lower bound) and 130 % (upper bound) with respect to the ordinates of the above-mentioned target spectra (Fig. 4).

Several groups of three accelerograms (one in longitudinal, one in transversal and one in vertical directions) for different IMs are considered in NLDAs and the maximum values of recorded results are taken into account for the generation of fragility curves.
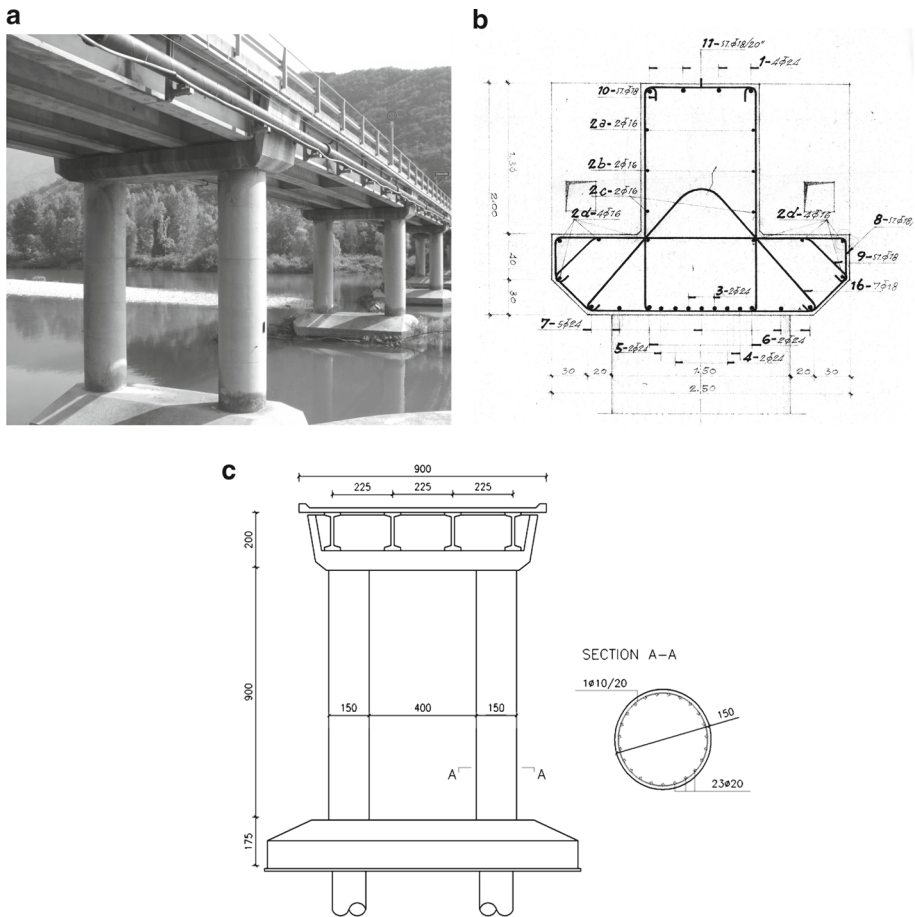
Fig. 5 Bridge case study geometrical characteristics: general view (**a**), original drawing of steel reinforcements of the pier top beam (**b**) and transversal section (c)

2.2.1 Step 2: Numerical example

The vulnerability assessment is demonstrated with a characteristic case study, originally built in the '70s (Fener bridge shown in Fig. 5a). The structure consists in a RC multi-span simply supported girder bridge. The bridge is 99 m long and it has 4 pre-stressed reinforced concrete (PRC) spans with double-tee beams and a cast-in-place RC slab (Fig. 5c). Each span is 24.75 m long. The spans are supported by RC framed piers with two circular columns (1.50 m diameter) and a transverse reverse-T beam (2 m high) as shown in Fig. 5b, which is taken from the original drawings. The piers are 9 m high and deck is 9 m wide. Each pier has a transversal framed structure, therefore it has a different behaviour in the two main directions (the static scheme is a cantilever beam in the longitudinal direction, whereas it is a framed structure in the transversal direction). Foundation structure is set up of plinths and four circular piles (1.25 m diameter and 16 m long) for each plinth.

In this study two main variables are considered to build fragility curves: unconfined concrete maximum stress f_{c0} and steel yielding strength f_y of the piers. Reinforcing steel is

Table 1 CFRP main materials' characteristics for the bridge case study retrofit intervention

|	
Young modulus (GPa)	230
Tensile ultimate stress (MPa)	3,430
Ultimate strain (%)	1.5
Density (kg/m^3)	1,820
One-layer thickness (mm)	0.165

FeB32K type with a lognormal probabilistic distribution (Mirza and MacGregor 1979). The mean value of the yielding strength is equal to 385 MPa and standard deviation is 42 MPa. This distribution is subdivided in three intervals of 82 MPa having the following central values: 303, 385 and 467 MPa. Concrete is supposed of class C25/30 according to CEN Comité Européen de Normalisation (2004) with a normal probabilistic distribution (Melchers 1999). The mean value of the unconfined concrete maximum stress is equal to 41 MPa and the standard deviation is 10 MPa. This distribution is subdivided in five intervals of 14 MPa having the following central values: 13, 27, 41, 55 and 69 MPa. Analogous considerations have been adopted for confined concrete.

Since a number of existing bridges need rehabilitation or strengthening because of improper design or construction, change of the design loads, damage caused by environmental factors or seismic events (Priestley et al. 1996; Kim and Shinozuka 2004; Zhou et al. 2010; Modena et al. 2014), retrofit interventions play an important role to increase seismic vulnerability of existing structures. Among different retrofit interventions (e.g., steel pier jacketing, concrete column jacketing, etc.), FRP technique has become a common and efficient technique for strengthening RC elements. In particular, FRP jacketing is commonly used to increase compressive strength and ductility of bridge piers. This hypothesis of pier jacketing is taken into account to evaluate benefits in terms of improvement of fragility curves.

Comparison of fragility curves for the entire bridge without retrofit interventions and the entire bridge with the FRP pier jacketing intervention is presented. In particular, the models described in National Research Council (2004) and Pellegrino and Modena (2010) are considered for the constitutive law of the FRP confined RC pier. With respect to the well-known model described in National Research Council (2004) adopted in the Italian Code, both the interacting contributions of existing internal steel reinforcement and external FRP strengthening are considered in the model developed by Pellegrino and Modena (2010). The retrofit intervention is made using four layers of carbon fiber reinforced polymer (CFRP) continuously wrapped along the height of the piers. The CFRP characteristics are reported in Table 1. CFRP characteristics are not considered as main stochastic variables (no probability density functions are associated to them) because CFRP material comes from a strictly controlled industrial production where variability is minimum.

Non-linear dynamic analyses are carried out by OpenSees code (McKenna et al. 2009) and piers are modelled with force-beam column elements. Kent and Park (1971) constitutive law, modified by Park et al. (1982), is considered for concrete elements and an elastic-hardening plastic law is used for reinforcing steel. The deck is modelled with linear elastic beam elements (Young modulus E equal to 34,760 MPa and shear modulus G equal to 14,480 MPa). After having verified that pier failure is due to flexural failure, the comparison is made for each Performance Level (PLs1 to 4) in both longitudinal and transversal directions. Results from longitudinal direction are shown in Fig. 6, while results from transversal direction and other investigations on the generation of fragility curves can be found in Morbin (2013).
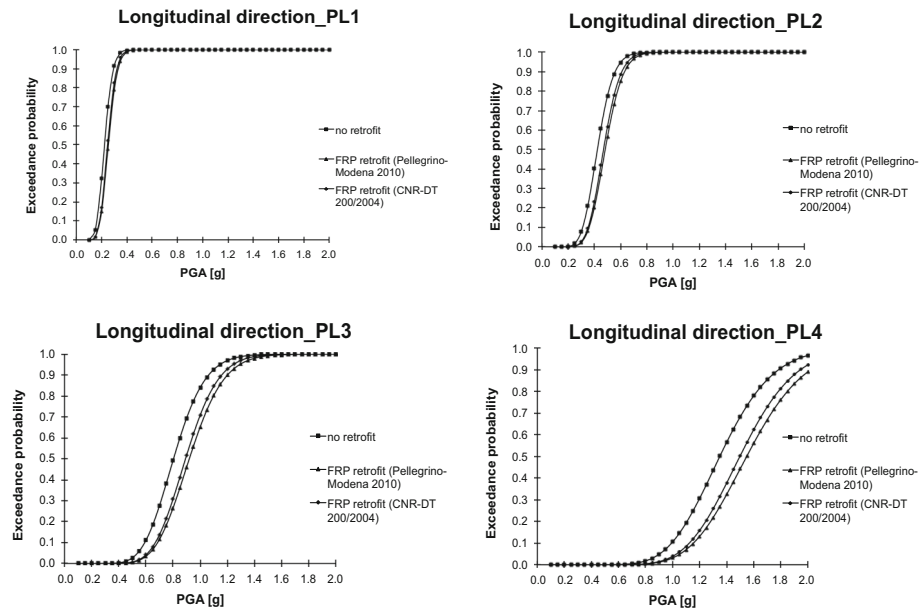
Fig. 6 Fragility curves in longitudinal direction: comparison between entire bridge without retrofit and retrofitted bridge using Pellegrino and Modena (2010) and National Research Council (2004) analytical models

2.3 Step 3: Inspections beginning

If an earthquake occurs, it is important to set a criterion when inspections on bridges can start. This issue is one of the most crucial key points in emergency management and it was often overlooked in previous literature studies. This work proposes the following criterion: inspections, or other NDE methods, start on a bridge if earthquake reaches or overcomes a specific seismic threshold intensity measure, expressed in terms of PGA or Sa[1s]. This threshold intensity depends on the definition of Performance Levels. In this study it is defined as the seismic intensity measure at which there is a 10 % seismic risk probability of observing the most vulnerable PL between moderate damage and shear failure, for a time period equal to the service life of the structure. This probability is calculated in the following equation, that defines the total PL occurrence probability (P_{PL}) in seismic risk assessment (Ellingwood and Kinali 2009):

$$P_{PL} = \sum_{IM} P\left(D > d_{PL}|IM\right) \cdot \left|\Delta H\left(IM\right)\right| \tag{6}$$

where H represents the hazard curve of the specific site.

Shear failure is considered herein because it is a brittle failure type and moderate damage (*PL2*) is considered because, according to Choi (2002), it is the first PL that affects structural stability, whereas light damage (*PL1*) is mostly a "cosmetic damage state". Moreover, the 10 % value (concerning seismic risk probability) is considered by analogy with life-safety limit state. According to Ellingwood (2009), life-safety limit state provided by building codes is considered the performance level that provides a good balance between uncertainty and risk acceptance. The threshold PGA values obtained for each bridge can be embedded in the database (Sect. 2.1) in order to perform faster interventions in emergency phase.
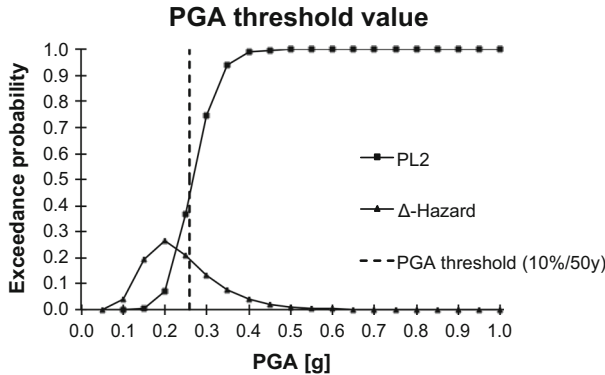
Fig. 7 *PL*2 50 years-10 % probability: PGA threshold value beyond which visual inspection can start
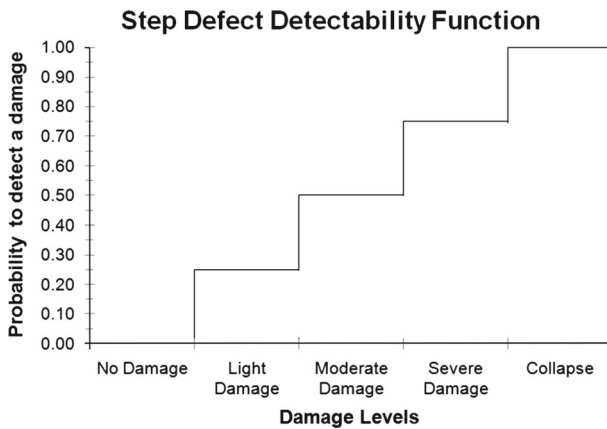

Fig. 8 The considered step defect detectability function for visual inspections

An example of this criterion, applied to the above-mentioned bridge case study (Sect. 2.2.1), is shown in Fig. 7. Moderate damage (*PL*2) fragility curve is considered to calculate PGA threshold beyond which visual inspections (or other NDE methods) can start. Solving Eq. (6), the threshold PGA value results 0.26 g (dashed line in Fig. 6).

2.4 Step 4: Inspection progress and fragility curves update

This step deals with the development of the inspections if the seismic intensity threshold is reached. Since this study considers piers as the most vulnerable elements in a bridge, inspections are referred to bridge piers. If the considered bridge has an unseated span or a pier is detected as collapsed, the bridge will be immediately closed. Otherwise, piers visual inspections and NDE investigations, are performed under a probabilistic point of view, considering uncertainties by analogy with a defect detectability function (DDF), as presented in Mori and Ellingwood (1994). Other studies (Ranf et al. 2007; Jerome and O'Connor 2010; Terzic and Stojadinović 2010; Alessandri et al. 2011) take into account inspections on bridges after a seismic event, but only from a deterministic point of view.

A step defect detectability function is considered in this study (Fig. 8). The probability to detect a damage is assumed to be equally subdivided between the damage levels:
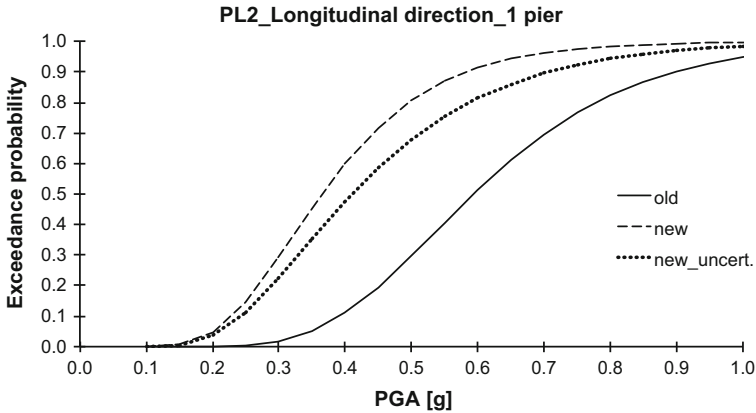
Fig. 9 Calculation of bridge case study fragility curves taking into account inspection uncertainties derived from the proposed methodology (*dotted curve*)

the smaller the defect is, the more the damage is difficult to detect with accuracy and the visual inspection is affected by this uncertainty. These damage levels are similar to the above-mentioned fragility curves performance levels, but conceptually different: the former are detected, the latter are predicted. The step defect detectability function damage levels are both visually and analytically described, as shown in Franchin and Pinto (2009).

2.4.1 Fragility curves update

After inspections, fragility curves have to be updated taking into account damages on the examined bridge, in relation to the aforementioned Light, Moderate and Extensive inspected damage states and the probability associated to each detected damage. Inspection of a pier is considered an independent event not related to other piers inspections of the same bridge or other bridges.

After generating the numerical model of the damaged bridge considering the detected damage levels, updated fragility curves are calculated by means of the procedures presented in Sect. 2.2, taking into account the same seismic action intensity of the fragility curves embedded in the database, as if the damaged bridge was subjected to another seismic event as strong as the artificial ground motion. Inspection uncertainty is considered as described in the sequence.

The probability of detecting a damage is considered as the probability that the updated fragility curve is not equal to the old (not updated) fragility curve. Therefore, calculating fragility curves for each pier of the considered damaged bridge, the above-mentioned probability to detect a damage is multiplied to the difference value between pier updated fragility curve and pier old fragility curve for each IM, reducing the gap and obtaining the updated fragility curve affected by inspection uncertainty (totally independent events). Subsequently, the fragility curve of the entire bridge is calculated via Eq. (4). These updated fragility curves replace the old ones in the database.

Considering the bridge case study in Sect. 2.2.1, we suppose that an earthquake occurred striking the three bridge piers and the damage, localized within 1 m from the pier base section, was detected as extensive damage (ED) for each pier. Figure 9 illustrates the updated

fragility curve, concerning one pier, *PL*2 and longitudinal direction, calculated by the proposed method.

2.5 Step 5: Allowing traffic

After updating fragility curves, taking into account inspection uncertainties, this subsequent step is focused on the calculation of a safety index in order to decide whether or not to allowing traffic over the examined bridge after a specific seismic event. Decision to closure of a bridge after a seismic event cannot be only based on visual (often rapid) damage inspections or observations without performing any analysis, because uncertainties govern the relation between damage patterns and loss of capacity (Franchin and Pinto 2009). These uncertainties are present even if monitoring instruments (that record displacements and damage on structures) are placed at each bridge and these real time data (together with prior analysis) are used to make a decision about the closure.

Mackie and Stojadinović (2006), adopting the Pacific Earthquake Engineering Research (PEER) approach, suggest a functional relationship which links reduction traffic volume and loss of vertical load-bearing capacity of bridge. However, calculation of loss of vertical load-bearing capacity related to a certain seismic intensity measure is affected by various important uncertainties.

It is of paramount importance to take into account the lack of literature on this topic and that is rather impossible to define both by means of inspection or monitoring systems and analytically without significant uncertainties related to the loss of vertical load-bearing capacity. For this reason, the proposed framework does not determine probability for partial traffic limitation and considers a suitable index to decide whether or not bridge is fully operational.

This bridge functionality index is defined in Eq. (7) considering the most vulnerable fragility curve between collapse (*PL*4) and shear failure. More specifically, a ratio between updated risk ($P_{PL,new}$) and old risk ($P_{PL,old}$) is calculated by analogy with Franchin and Pinto (2009):

$$\frac{P_{PL,new}}{P_{PL,old}} \leq 1 \rightarrow \text{Fully Operational Bridge} \tag{7}$$

where $P_{PL,new}$ is the updated risk calculated from Eq. (6), considering new (updated) fragility curve with inspections uncertainties (Sect. 2.4.1), whereas the $P_{PL,old}$ is the maximum value between the old (pre-earthquake) risk of the considered bridge and the average pre-earthquake risk among the bridge population of that seismogenic zone (SZ).

If $P_{PL,old}$ of the considered bridge is larger than the average pre-earthquake risk among the bridges of the whole network, then this bridge needs a seismic retrofitting. However, since the bridges on the considered SZ were open to traffic before the seismic event, it is reasonable to consider their average pre-earthquake risk in terms of allowing traffic, since this risk level was (indirectly) accepted by the bridge operating authorities.

If the index in Eq. (7) is larger than 1, the bridge is not fully operational and, not considering partial traffic limitations, the bridge can be opened for emergency operators or totally closed. Certain index values for the bridge not fully operational are suggested in Franchin and Pinto (2009): e.g., 3–5 the bridge can be openned for emergency operators. These values are affected by variability and uncertainty that depend on different aspects about the single bridge and the characteristics of the considered SZ. Therefore, in this study bridge can be considered only opened (fully operational) or closed (not fully operational), according to Eq. (7).
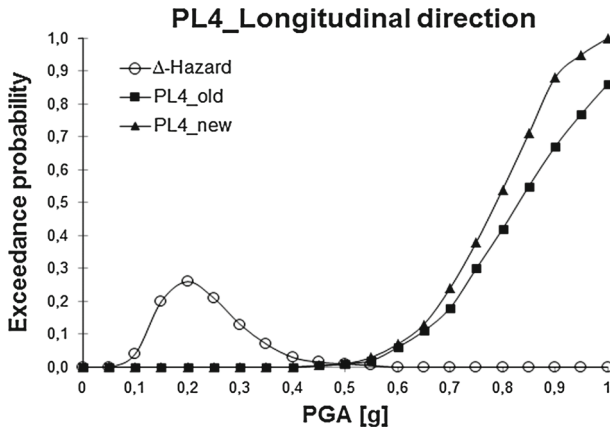
Fig. 10 *PL*4 fragility curves and Δ-Hazard curve for the analyzed bridge case study (Sect. 2.2.1)

It is stressed that this criterion can be always considered valid in emergency and post-emergency phases after a seismic event. More investigations about bridge load-bearing capacity can be carried out in following periods and the bridge could be opened without any retrofitting intervention, in accordance with bridge authorities and engineers investigations.

Concerning the bridge case study (Sect. 2.2.1) and the inspected damages as described in Sect. 2.4.1, *PL*4 (longitudinal direction because the most vulnerable) old and new fragility curves and Δ-Hazard curve (Sect. 2.3) are considered to calculate the seismic risk probability as in Eq. (6) (Fig. 10). The results are as follows:

$$P_{PL,old\ bridge} = 0.18\,\%$$
$$P_{PL,new\ bridge} = 0.42\,\%$$

The average pre-earthquake risk among the bridge population of that SZ is calculated from a previous study regarding seismic vulnerability of bridges in that area (Grendene 2006):

$$P_{PL,old\ bridge\ SZ} = 0.34\,\%$$

Since $P_{PL,old\ bridge\ SZ}$ is larger than P_{PL,old_bridge}, the ratio in Eq. (7) is equal to 1.23, therefore the bridge is suggested to be closed.

2.6 Step 6: Costs probability evaluation

The last step of the proposed framework deals with the estimation of repair costs to be sustained for damaged bridges. This step is mostly important for public authorities and transportation network operators. A number of studies in literature take into account calculations of various costs after a seismic event under different perspectives. For instance, Mackie and Stojadinović (2006) considers a component-level decision for repair cost based on limit states (PEER approach), Franchin et al. (2006) concerns costs evaluation in terms of human losses, Zhou et al. (2010) considers costs for the entire road network, mostly in terms of social cost (traffic delay, etc.). Although these approaches are important for the evaluation of the entire road or railway network, evaluation of costs of a single bridge in the network is equally important.

Herein extra attention is given to repair cost (C_{repair}), in particular when it is more expensive than replacement cost ($C_{replace}$), and this aspect is taken into account under a probabilis-

Table 2 Repair and replace unitary costs estimation for the considered bridge case study

PL2	$C_{repair\ FRP,\ PL2}$ (€/m^2)	300	1 CFRP layer
PL3	$C_{repair\ FRP,\ PL3}$ (€/m^2)	600	3 CRFP layers
Replace	$C_{replace}$ (€/m^2)	900	Bridge reconstruction

tic point of view. Concerning that piers are considered the most vulnerable elements of the bridges and the use of FRP composites has become common among seismic retrofit techniques, the repair cost is referred to FRP retrofit interventions on the piers as representative of the entire bridge damage (Mackie and Stojadinović 2006).

Firstly, FRP retrofit interventions cost ($C_{repair\ FRP}$) needs to be defined. This cost mainly depends on piers geometry and the number of FRP layers. It's supposed that seismic retrofit interventions are made, as previously described, by means of CFRP layers, continuously wrapped along the height of the piers. In the sequence, in accordance with fragility curves PLs, CFRP retrofit interventions are estimated for *PL2* and *PL3*. *PL1* and *PL4* are neglected because *PL1* is mostly a "cosmetic damage state", thus C_{repair} is always more economical than $C_{replace}$, whereas *PL4* represents the collapse of the bridge, therefore other intervention typologies are needed. The implemented retrofitting consists of one CFRP layer for *PL2* and three CFRP layers for *PL3*.

After having conducted a cost analysis considering different Italian regional pricelists, the following costs have been used: 300€/m^2 for the first CFRP layer and 150€/m^2 for the other layers. Furthermore, a range of 750–1,200€/m^2 for replacing an ordinary RC multi-span simply supported girder bridge, depending on foundations, piers geometry, deck, etc. In this work a mean unitary replace cost has been assumed. The considered unitary repair and replace costs are reported in Table 2.

Repair costs and replace cost (for each PL and each bridge) are compared to determine the most expensive repair cost by the following cost ratio (C_r):

$$C_r = \frac{C_{repair}}{C_{replace}} \tag{8}$$

If $C_r > 1$ repair cost exceeds replacement cost, whereas replacement cost is as expensive ($C_r = 1$) or more expensive ($C_r < 1$) than repair cost.

Since repair cost is related to PLs, it is affected by uncertainty, which is calculated by seismic risk associated to the PLs by Eq. (6). Updated fragility curves (Sect. 2.4.1) and Δ-Hazard curve (Sect. 2.3) are considered and the seismic risk value is strictly related to the results in Eq. (8).

The proposed criterion is mainly applied for fully operational (damaged) bridges. Considering updated fragility curves, costs predictions refer to bridge conditions after another seismic event. Hence, it is useful for bridge operators in order to decide whether or not to retrofit the bridge before another seismic event occurs. Then, the decision to make seismic retrofit interventions depends on different aspects: budget availability (at that moment), bridge importance within the road or railway network, acceptance of the risk from public authorities, etc. Taking into account not fully operational (damaged) bridges, this criterion could be applied if it is decided to open the bridge after further investigations to assess its structural integrity.

Concerning the bridge case study in Sect. 2.2.1 and the relative results in Sect. 2.5, this criterion is developed. Since it is suggested that the bridge should be closed (not fully oper-

Table 3 Repair and replacement costs for the considered bridge case study

		Unitary costs	Piers surface	Repair costs	Cost ratio
PL2	$C_{repair\ FRP,\ PL2}$	300 (€/m^2)	254.34 (m^2)	76,302 (€)	0.10
PL3	$C_{repair\ FRP,\ PL3}$	600 (€/m^2)	254.34 (m^2)	152,604 (€)	0.19
			Deck area	Replacement cost	
Replace	$C_{replace}$	900 (€/m^2)	891 (m^2)	801,900 (€)	

ational), it is assumed that the bridge is re-openned after further rapid investigations. The considered surfaces to calculate costs (CFRP is continuously wrapped along the height of the circular piers columns), total repair costs for *PL*2 and *PL*3, reconstruction cost and finally cost ratios of the two repair alternatives are shown in Table 3.

Cost ratios highlight that retrofitting the bridge is cheaper than replacing it, for both *PLs* considered. Finally, *PL*2 and *PL*3 seismic risks are calculated considering updated fragility curves (Sect. 2.4.1) and Δ-Hazard curve (Sect. 2.3), obtaining the following results:

$$P_{PL2,new} = 58.3\,\%$$
$$P_{PL3,new} = 28.1\,\%$$

These results mean that, if a 475 years return period seismic event occurs, there is a probability of 58.3 % to spend €76,302 and a probability of 28.1 % to spend €152,604 for retrofitting the bridge. It is noticed that these costs are much lower than bridge replacement cost, i.e., 10 and 19 % of the estimated replacement cost. Considering this evaluation and the above-mentioned motivations regarding road network, budget availability, importance of the structure, etc, public authorities can decide to accept these risks and not immediately retrofit the bridge or, in contrast, they can decide to make interventions on the bridge in order to reduce these risks.

3 Conclusions

This study presents a complete probabilistic framework for the seismic assessment of existing bridges in order to provide useful information to bridge authorities or not to allow traffic over a bridge after a seismic event and to immediately retrofit damaged bridges. This procedure is mainly based on analytical fragility curves and on bridge visual inspections (non-destructive evaluation methods) considered under a probabilistic point of view by means of a defect detectability function. A particular focus is given to common FRP retrofit interventions for RC pier confinements, and their impact on seismic vulnerability reduction. Moreover, the whole probabilistic procedure is applied step by step to a RC bridge case study having a common structural typology in Italy (multi-span, simply supported girder bridge).

The following key points characterize the original contribution of this paper:

– A fragility curves updating procedure is conceived taking into account visual inspections on the damaged bridge and uncertainties associated with them. A damaged bridge is seismically more vulnerable than an undamaged one, as shown in Fig. 5. It is stressed that damaged bridge fragility curves depend on inspected damage, which modifies the stiffness of the entire structure and, so, its fundamental period. In particular, it's well-known that damaged structure stiffness is lower than for the undamaged structure, therefore the

proper period of the damaged structure could shift to low spectral acceleration which entail minor seismic loads. These two aspects can make damaged structure fragility curves not always more vulnerable than undamaged ones.

– A criterion is proposed to consider bridge fully operational or not fully operational, without accounting any partial traffic limitation. Other studies, e.g., Mackie and Stojadinović (2006), calculate the loss of vertical load-bearing capacity related to seismic intensity measure in order to consider partial traffic limitation, but the procedure is affected by important uncertainties in all the phase of the process. For this reason, a simplified traffic limitation criterion is proposed.

– A retrofit intervention cost assessment procedure is proposed aiming to provide useful and concurrently simplified indications for potential benefits if the bridge is struck by another earthquake, in particular if the bridge repair costs can exceed replacement costs. This cost assessment process is performed under a probabilistic point of view, since it is associated to a specific seismic risk. Decisions whether or not retrofitting or replacing the bridge have to be taken with respect to other aspects as economical budget availability, importance of the structure, risk analysis of the whole road network, etc. In any case, such cost indications are useful for proper budget allocation.

Appendix

In the following, the main steps for the construction of the fragility curves for a bridge, according to the above-mentioned procedure, are presented.

1. Assemblage of a group of accelerograms compatible with the elastic spectrum of the site of interest. In this study, according to the Italian Code for Constructions (Italian Ministry of Infrastructures 2008), seven artificial accelerograms were considered for the analysis of the structure in longitudinal and transverse direction. Each artificial accelerogram is scaled by a numerical factor to obtain various values of peak ground acceleration (PGA) to perform the fragility analysis.

2. Generation of statistical samples of the bridge considering significant modelling parameters. Accordingly, two main variables have been considered for the pier: steel yielding strength f_y and unconfined concrete strength f_c. A probability density function is associated to each variable. These functions are subdivided into finite intervals to match the intervals and make nominally identical bridge samples, but statistically different.

3. Non-linear time history analyses are developed for each ground motion-bridge sample. In this study, displacement on pier top was monitored throughout the analyses.

4. For each run, peak responses in longitudinal and transversal directions were recorded in order to calculate the damage as shown in Eq. (5). These results were plotted versus the value of the intensity measure for that ground motion in a bi-logarithmic plane (see Eq. 2). A linear regression of these data is then used to estimate A and B coefficients, medium value and dispersion.

5. The fragility curve for a significant bridge component (e.g., the pier), at a certain Performance Level and direction (longitudinal or transversal) can be calculated numerically solving the integral:

$$\int_{D(a)>dPL} f_D(d/a)\partial d \qquad (9)$$

by means of Eq. (3) and the procedure described in Sect. 2.2.

6. Finally, the fragility curve of the entire bridge for each Performance Level can be calculated via Eq. (4).

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