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ORIGINAL RESEARCH PAPER

Life-cycle cost optimization of the seismic retrofit of existing RC structures

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Abstract The life-cycle cost analysis of buildings prone to seismic risk is a critical issue in structural engineering. Expected loss, including damage and repair costs, is an important parameter for structural design. The combination of economic theory and computer technology allows for a more developed approach to the design and construction of structures than ever before. In this study, a simplified method based on a semi-probabilistic methodology is developed to evaluate the economic performance of a building prone to seismic risk. The proposed approach aims to identify the most cost-effective strengthening strategies and strengthening levels for existing structures during their structural lifetime. To achieve this, the method identifies the optimal strengthening level, computing on the one hand the costs of strengthening the structure at different performance levels for each strategy, and, on the other, the expected seismic loss during the structure's lifetime. To assess the expected loss, the building is divided into several components, both structural and non-structural. A set of fragility curves is assigned for each component. Then, once the structural model and the various components of the building, with the corresponding fragility curves, are defined, a loss assessment is performed using a static non-linear analysis. The summation of the strengthening costs and the discounted expected losses produces a relationship between the total costs and the strengthening level. The minimum of this relationship identifies the most cost-effective strengthening intervention. As a case study, this method is applied to an existing reinforced concrete (RC) structure severely damaged by the 2009 earthquake in L'Aquila. Different strategies are analyzed, namely the FRP (fiber reinforced polymer) strengthening of elements, the RC jacketing of columns, RC exterior shear wall insertions, and the base isolation of the building.

Keywords Life cycle cost assessment · Seismic assessment · Economic sustainability

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1 Introduction

In a very seismic country such as Italy, it is evident how attention has focused in recent years on seismic design, with a view to guaranteeing an adequate structural performance with the purpose of safeguarding human life. However, developments in relation to the April 6 2009 earthquake in Abruzzo have also shown that the economic losses suffered by buildings linked to the earthquake are issues of great importance (Di Ludovico et al. 2016a, b).

In the construction industry, decision-making with respect to structural and non-structural systems situated in seismic areas requires consideration of the damage and other costs resulting from possible earthquakes during the lifetime of a structure. Accordingly, the life-cycle cost assessment (LCCA) procedure can become an essential component of the design process in order to control the initial and future costs of a building (Lagaros 2010). In fact, this procedure is based on consideration of initial costs and lifetime costs over the structure's lifetime (Wen and Kang 2001).

One of the first building loss estimation methodologies was advanced by Scholl et al. (1982), who developed and suggested improvements to both empirical and theoretical loss estimation procedures. Part of the theoretical research included an in depth study of developing damage functions for a variety of building components based on experimental test data. This proposed breaking down a building into various components and predicting the damage caused to each of them as a function of seismic intensity. The purpose of the study was to calculate the damage factor, which was defined as the ratio between the cost of the damage caused by an earthquake and the cost of replacing a building.

The method proposed by Scholl et al. (1982) required component damage functions to estimate damage to a building component. In conjunction with the Scholl et al. (1982) study, Kutsu et al. (1982) collected laboratory test data to estimate damage to various building components in order to implement the proposed component-based methodology. The components evaluated included both structural members (beams, columns, and shear walls) and non-structural components (masonry walls, drywall partitions, and glazing). Using these laboratory tests, it was possible to derive a relationship between the intensity of an earthquake and the damage to each component, and thus the cost of the construction. This type of assessment was, however, carried out with an elastic analysis, and cannot therefore represent the real state of damage to a structure when it is affected by the plasticization phenomena.

A more detailed loss estimation methodology was introduced by Gunturi and Shah (1993). Structural behavior was evaluated with a non-linear analysis, with different ground-motion records applied to a building's foundations. The building was divided into structural and non-structural elements, and the damage was calculated by obtaining structural response parameters for each non-linear time history analysis.

The variability in ground motion as it relates to assessing economic losses for buildings was addressed in a study by Singhal and Kiremidjian (1996). A systematic approach to developing motion-damage relationships was proposed by subjecting a structure to a suite of simulated ground motions, and obtaining its probabilistic response using a Monte Carlo simulation.

Porter and Kiremidjian (2001) introduced an assembly-based probabilistic loss estimation methodology that accounted for more sources of uncertainty than previous studies. The study also incorporated the uncertainty of estimating the damage to each component and the ambiguity associated with estimating repair costs as a function of this damage.

A Monte Carlo simulation was used in this framework to predict building-specific relationships between expected loss and seismic intensity. To predict losses for an application case, techniques for developing fragility models for common buildings were presented.

As members of the Pacific Earthquake Engineering Research (PERR) center, Aslani and Miranda (2005) developed a methodology that incorporated the influence of collapse on monetary loss by estimating the probability of collapse at different levels of ground motion intensity. However, losses due to building demolition were not included in the evaluation of expected seismic losses. This component-based methodology also proposed approaches for disaggregating buildings into components in order to estimate which were the most significant in terms of influencing total losses.

Zareian and Krawinkler (2006) proposed a simplified version of the Aslani and Miranda framework. This approach used a semi-geographical method to evaluate the economic loss component. In particular, the approach evaluated economic losses by grouping components into subsystems (at either the storey or building level). Components of the same subsystem were then represented by a single engineering demand parameter.

LCCA has been implemented also for the assessment of the European seismic design codes and in particular EC2 and EC8 with respect to the recommended behaviour factor q. The assessment is performed on a multi-storey RC building which was optimally designed (Lagaros 2010).

Recently, several studies have focused on the assessment of building repairability via the estimation of expected performance losses and associated costs of repair and, if necessary, the cost of strengthening existing RC buildings. In this case, it is necessary to establish if it is more convenient to repair and retrofit or to demolish and rebuild (Holmes 1996; Polese et al. 2013, 2015; Di Ludovico et al. 2013). Life cycle cost assessment (LCCA) procedure can be considered an essential tool for the design process in order to control the initial and the future cost of building ownership.

Padgett et al. (2010) proposed also a method for evaluating the best retrofits for non-seismically designed bridges based on seismic life-cycle costs and cost–benefit analysis.

Kappos and Dimitrakopoulos (2008) implemented decision making tools, namely cost-benefit and life-cycle cost analyses, in order to evaluate if a pre-earthquake strengthening of a large and heterogeneous building stock is feasible or not, and what is the optimal retrofit level for mitigating the seismic risk. In addition a cost-benefit and life-cycle cost analysis has been carried out by Chrysostomou et al. (2015) to evaluate the effectiveness of a strengthening programme adopted in Cyprus and to evaluate the optimum retrofit levels for each building type examined. Moreover, authors aim to provide a kind of guide for any future strengthening programme of important buildings characterised by unacceptable level of earthquake risk. Also Liel and Deierlein (2013) evaluated mitigation alternatives for older concrete frame building through a cost-benefit assessment. The present study aims to provide a simplified methodology for practitioners to use to assess the most cost-effective intervention strategy for existing structures by means of an economic and seismic capacity performance evaluation in a structure's life-time. The goal is to determine the optimal intervention to balance the seismic safety level and the expected seismic losses in the structure's life-cycle.

2 Proposed methodology

This section describes the methodology for performing a seismic capacity assessment of a structure in its original and strengthened configuration, and for evaluating the economic performance during the structure's life-time. The methodology proposed herein is based on

the PEER's approach, but this section also underlines the differences between the two methods.

2.1 PEER approach

Performance-based earthquake engineering (PBEE) consists of the evaluation, design, and construction of structures prone to seismic risk. Different measures of seismic performance can be selected in a PBEE framework, such as economic loss, death, and the time a facility is unavailable. The most commonly used PBEE approach for the assessment of a life-cycle cost analysis is the "PEER's methodology" developed by the Pacific Earthquake Engineering Research body (Porter 2003).

The most relevant advantage of this approach is that it also incorporates the uncertainty resulting from the estimation of damage to a construction and the associated repair costs. This methodology is wholly probabilistic and consists of the numerical integration of all the conditional probabilities propagating the uncertainties from one level of analysis to the next (Goulet et al. 2007).

Figure 1 schematically shows the PEER methodology, which works in four stages: hazard analysis, structural analysis, damage analysis, and loss analysis. Their outputs are, respectively, the intensity measure (IM), the engineering demand parameters (EDPs), the damage measure (DM), and the decision variable (DV). The expression p[X|Y] refers to the probability density of X conditioned on knowledge of Y, and g[X|Y] refers to the occurrence frequency of X given Y (Porter 2003).

Consequently, the PEER framework equation is:

$$g[DV|D] = \iiint p[DV|DM]p[DM|EDP, D] \\ p[EDP|IM, D]g[IM|D]dIMdEDPdDM \tag{1}$$

where g[DV|D] is the mean annual probability that the DV exceeds a specific value given a facility, p[DV|DM] is the conditional probability that the DV exceeds a specific value of the DM, p[DM|EDP,D] is the derivative (with respect to the DM) of the conditional probability that the DM exceeds a limit value given a value of the EDP, p[EDP|IM,D] is the derivative of the conditional probability that the EDP exceeds a limit value given a value of the earthquake IM, and g[IM|D] is the derivative of the seismic hazard curve given a site location.

In the hazard analysis, the mean annual rate of exceedance of a particular ground-motion IM at the facility site is evaluated, assuming Poisson distribution model of earthquake occurrence.

In the structural analysis phase, an Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell 2002) is performed to evaluate the response of the facility to the ground motion of a given IM in terms of inter-storey drift, peak floor acceleration, peak plastic hinge rotation or other EDPs. Each ground motion is scaled in increasing intensity until the onset of structural collapse. The IDA study is implemented through the following steps:

(i) define the nonlinear FE model required for performing nonlinear dynamic analyses; (ii) select a suit of natural records; (iii) select a proper intensity measure and an engineering demand parameter; (iv) employ an appropriate algorithm for selecting the record scaling factor in order to obtain the IDA curve performing the least required nonlinear dynamic analyses and (v) employ a summarization technique for exploiting the multiple records results (Lagaros 2010). Selecting IM and EDP is one of the most important steps of
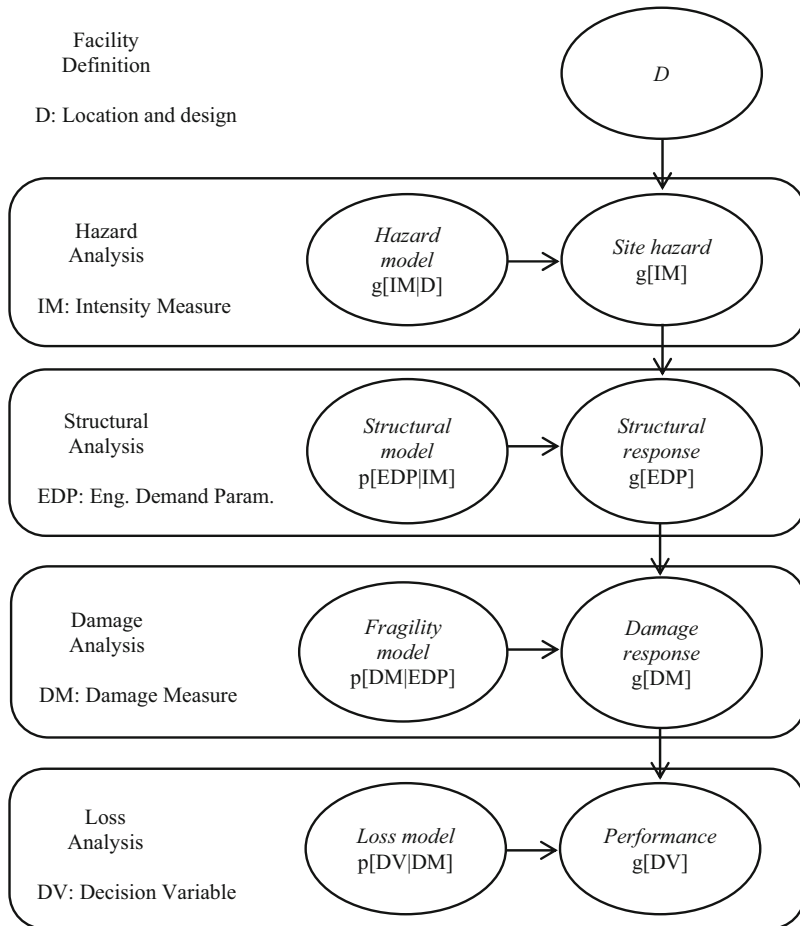
Fig. 1 PEER analysis methodology

the IDA study. The EDPs are classified into four categories: engineering demand parameters based on maximum deformation, engineering demand parameters based on cumulative damage, engineering demand parameters accounting for maximum deformation and cumulative damage, global engineering demand parameters.

The third step, the damage analysis, uses the EDPs with component fragility curves to estimate the probability that a component is in, or exceeds, a particular damage state. Once the damage state of a component has been estimated, it is possible to evaluate the repair efforts needed to restore the component, the relevant repair costs, operability, and the repair duration. These measures of performance are used in the fourth step to establish the probabilistic losses.

The first to implement this method for evaluating the seismic damage to a building were Aslani and Miranda (2004). Their study, in agreement with the PEER's methodology, assessed the economic performance of a building, taking into account the inter-storey drift and the acceleration of the top of the building as a parameter of the structural response.

This procedure could, however, be complicated, because of the type and amount of the required computations. This is why subsequent studies have been directed towards a simplification of the PEER's methodology in order to reduce the amount of information required or the time involved in performance estimations. This idea was backed up by the work of Ramirez and Miranda (2009), who tried to develop a more simplified process than their predecessors. In their study, they proposed an approach which, starting from the same basic principles of the PEER's methodology, reduces the amount of data that a designer must consider during the computations. This may be possible by introducing the functions which relate response simulation data directly to economic losses (EDP-DV functions).

The EDP-DV functions were also developed to estimate the damage to a component that does not have an appropriate fragility model using generic fragility functions based on empirical data.

2.2 Assessment of economic losses according to the proposed approach

In this study, a simplified semi-probabilistic methodology is proposed to assess easily the economic performance of a building prone to seismic risk.

The approach developed consists of the same steps as the PEER's methodology.

The first step is site hazard characterization, which is developed fully in a probabilistic way. Ground motion hazard characterization involves the quantification of an earthquake's IM. The probability of exceeding the intensity of a given earthquake can be evaluated in a simplified manner that is equal to the inverse of the return periods, T_r. In fact, the Italian code contains nine return periods for each site, and the nine pieces of data can be assumed to be the range of eight observation time intervals. Each interval is represented by the probability of the occurrence of a generic earthquake with a return period between two consecutive return periods set out in the code. The following formulation can be used to quantify the probability of occurrence of an earthquake with an intensity belonging to a certain range of return periods:

$$p_{r,i}\left(T_{r,i} < T_r < T_{r,i+1}\right) = \frac{1}{T_{r,i}} - \frac{1}{T_{r,i+1}} \tag{2}$$

where the subscripts i and $i + 1$ define two consecutive return periods, and T_r, and $p_{r,i}$ is the probability of occurrence of a certain return period.

The structural analysis step in the PEER's methodology is simplified here by means of a static non-linear analysis instead of a non-linear time-history structural analysis. The use of a non-linear static analysis makes the methodology suitable for common applications. Furthermore, such an analysis is commonly carried out by practitioners to assess the seismic capacity of existing structures and design strengthening interventions. This choice results in an average evaluation of the structural response given the intensity of the seismic event. So, formally, in Eq. (1), the term p[EDP|IM,D] is not introduced, since the average structural response is identified for each seismic intensity. To do this, a static non-linear analysis is carried out on the structure up to its global mechanism. The bilinearization procedure is performed according to the N2 approach for each step of the pushover curve (Fajfar 1999). Accordingly, a PGA value is derived for each step of the pushover curve as the demand intensity that would induce that particular structural response. It is possible to assume an average structural response for each hazard intensity, defined in terms of the PGA. The simplification holds in the fact that, given each deformation pattern of the structure at the different steps, a set of mean values for the EDPs is obtained. In other

words, given the value of the top displacement that controls the pushover curve associated with each hazard intensity (in terms of the PGA), all the values for all the EDPs of interest are on average derived (e.g. IDR and the Spectral Acceleration, SA, on each floor). It is possible to identify a PGA value corresponding to the maximum top displacement of the curve. According to this approach, this value is assumed to be the hazard intensity that would induce the structural failure by activating the collapse mechanism. For each hazard intensity value equal to or greater than this, the occurrence of the structural collapse is on average assumed. In this case, there is no need to pass through the fragility models of each component for the derivation of the damage, and the economic loss is assumed to be equal to the overall reconstruction costs.

In the third step, to assess the damage to the building components, a set of fragility models are used providing, through the parameters of the structural response, the probability of occurrence of a certain level of damage. The building is divided into various components, both structural and non-structural, and for each of these a set of fragility curves is assigned that is representative of a certain intensity of damage. Therefore, more than one fragility curve can be assigned for each component, corresponding to a level of damage that is gradually greater. In detail, the EDPs that control the damage to each component are derived from the output of the structural analysis, and are used as an input to the fragility models in order to estimate the probability of occurrence of each damage state.

So, in order to convert the damage to a component into a contribution to the economic losses of the building, it is necessary to compute the cost of each repair/recover intervention from the damage level or substitution. In fact, for each fragility curve, the damage state corresponds to the economic effort needed to restore the component to an undamaged state. This allows us to assess the economic losses of the entire building as the sum of the repair/recovery costs of each component multiplied by the probability of occurrence.

In other words, the yearly economic losses of the building can be computed as:

$$EAL = \sum_{i=1}^{n} \sum_{DS_j} \overline{C}_{i,DS_j} \int p\left[DS_j|\overline{EDP_j}(IM)\right] g[IM|D] dIM \qquad (3)$$

where n is the number of the building components; DS_j is the j-th damage state of the fragility model of a component; C_{i,SD_j} is the cost to restore the component i due to the damage state DS_j; $\left[DS_j|\overline{EDP_j}(IM)\right]$ is the probability of occurrence of the damage state DS_J for the i-th component given the EDP that depends on the intensity measure; $g[IM|D]$ is the derivative of the seismic hazard curve given a site location.

The difference with Eq. (1) is in the absence of the derivative of the conditional probability that the EDP exceeds a limit value given a value of the earthquake's IM. Finally, the economic loss calculated according to Eq. (3) is computed over the life-time of the building and multiplied by the discount rate in order to actualize the total losses. Present-value discounting accounts for the time-value of money, recognizing that money paid or earned today is valued more than the same amount in the future. The discount rate is determined from interest rates and adjusted for inflation, and traditionally ranges from 2 to 6% (Nuti and Vanzi 2003). This can be calculated using the following equation:

$$D_r = \sum_{i=1}^{V_n} \left(\frac{1}{1+d}\right)^i \qquad (4)$$

where d is the value of the yearly discount rate and V_n is the life-time of the structure.

For further clarification is necessary to point out that this approach is significantly different from Vamvatsikos and Cornell (2002) in which the structural model is transformed into a SDOF system and subjected to one (o more) ground motion record(s), scaled to multiple levels of intensity, thus producing one (or more) curve(s) of response parametrized versus intensity level. The proposed approach is much similar to the Incremental N2 (IN2) method proposed by Dolsek and Fajfar (2004). This method is a simple tool and can be employed for the determination of the approximate summarized IDA curve. The seismic demand is determined for multiple levels of seismic intensity using the N2 method (Fajfar 1999) (based on pushover analysis and inelastic response spectrum). The quantities used to represent the intensity measure and the engineering demand parameter are the spectral acceleration at the natural period of the equivalent single-degree-of-freedom (SDOF) model and the top displacement. The IN2 curve can substitute the IDA curve in the probabilistic framework for seismic design and assessment of structures. Authors also show a reasonable accuracy of the IN2 curve in comparison with the IDA curve for the examples adopted (Dolsek and Fajfar 2007). The dispersion measures for randomness parameters β_i cannot be determined from the results of the IN2 analysis and are predetermined.

On the contrary, in the approach here proposed the randomness parameters for the structural response β_i have not been introduced since the final scope of the procedure is the evaluation of the expected annual loss. Thus, both for the structural response, given the intensity measure, and for the replacement costs of the components, given the damage limit state, only the average values have been used. This approach can be interpreted as a further simplification of the IN2 method where the PGA is used as intensity measure instead of the Spectral Acceleration. Moreover, authors do not use SDOF model to perform dynamic analysis, but to assess damage levels on building components and the expected economic loss.

2.3 Optimization of strengthening interventions

The proposed methodology aims to identify the most cost-effective strengthening strategies and strengthening levels (i.e. strengthening intervention associated with a given safety level) for existing structures in a structure's life-time. Indeed, the structural analysis could show a very low safety level for the structure in the original configuration and a strengthening intervention could be necessary. The safety level, expressed as a percentage, represents the ratio between the capacity of the structure, the PGA capacity, and the demand of the quake, namely the PGA demand. Analyzing the pushover curve step-by-step, a PGA value can be associated with each step. The PGA associated with a failure is defined as the PGA capacity. A safety level of 100% means that, once strengthened, the building has achieved a safety level equal to that required of a new building designed according to current seismic code provisions.

In order to determine the most cost-effective strengthening solution, it is necessary, once the intervention strategy is identified, to calculate on the one hand the costs of strengthening the structure and, on the other, the expected seismic losses in the structural life-time at different performance levels (i.e. safety levels). In particular, each performance level corresponds with a level of strengthening intervention and relevant costs. Therefore, the cost of strengthening the building for various safety levels is obtained. The result will be a curve of costs that increase with the increase of the strengthening actions.

Both of the interventions that increase structural stiffness (and thus limit displacements), and those that increase ductility, generate a potential level of damage to the

structure in its life-time that is lower than that which would occur to the structure if it were not strengthened. The goals are: to assess whether the cost of the strengthening is beneficial enough to justify the intervention in the structural life-time of the building, and to identify the optimal strengthening level. For each safety level, the sum of the costs of the strengthening interventions and the economic loss associated with such a safety level is called the "expected total cost" for the building. The maximum safety level corresponding to the lowest value of the expected total cost will represent the most cost-effective solution, as set out in Fig. 2. This figure reports three curves: (1) the "economic loss" curve, which represents the economic losses related to several safety levels (the first point of the curve is the economic loss if no strengthening interventions are made; for the sake of simplicity in Fig. 2, this point is related to a very low safety level, as commonly found in existing structures); (2) the "cost of the strengthening intervention" curve, which reports the costs required to attain a given safety level; and (3) the "expected total cost" curve, which is the sum of the costs reported in the previous curves associated with each safety level.

The curves are a schematic representation of the methodology. It is worth nothing that the curves may be determined for different strengthening strategies involving different strengthening techniques in order to identify the most cost-effective strengthening solution. The cost-effectiveness of retrofitting is highly dependent on the cost of the retrofit, the level of strengthening, the seismicity of the region, and the time horizon considered (Liel and Deierlein 2013).

Using this procedure, it is possible to provide practitioners an additional tool to quickly evaluate what is the best decision to make concerning an existing building from an economic point of view. It should be noted that the best choice from an economic point of view could not achieve an adequate safety level, meaning that the safety level required can also be selected according to code provisions and as a balance between risk and costs.

As a summary, Fig. 3 shows the scheme of the proposed methodology divided into seven simple steps. The first and second steps involve a suite of static non-linear analyses of the strengthened structure using several strategies aimed at achieving target security levels (risk indices). The fourth, fifth, and sixth steps concern the cost of the strengthening interventions, the total expected cost, and, thus, the most cost-effective level of strengthening for each strategy. Finally, the seventh step identifies the most cost-effective intervention strategy.
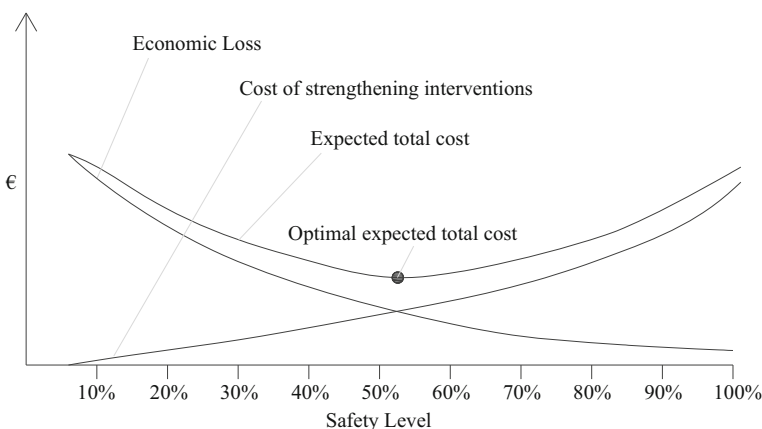

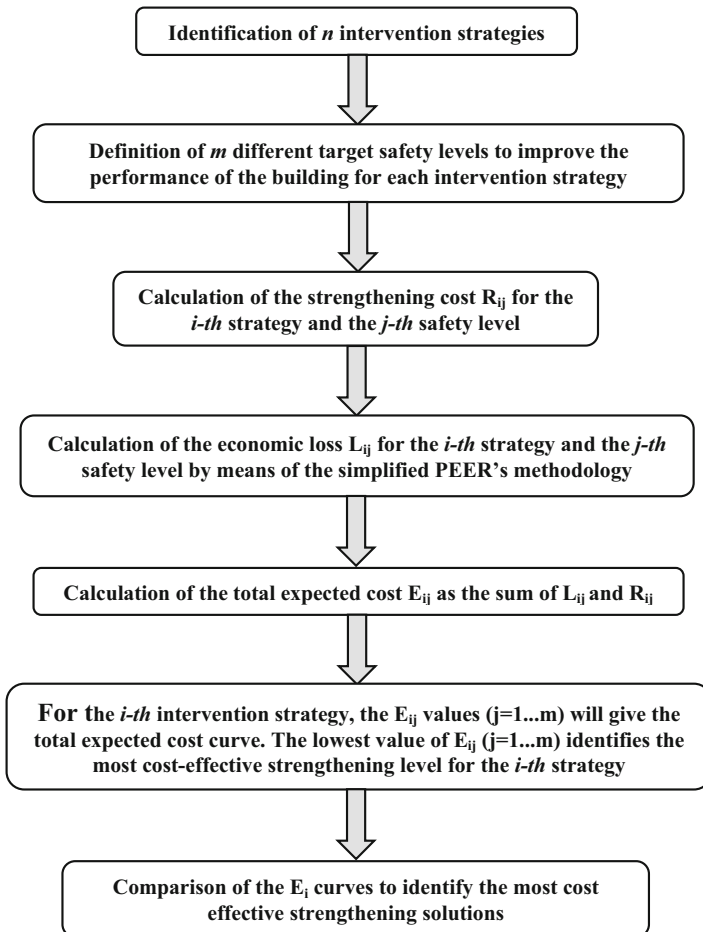**Fig. 2** Procedure for the strengthening optimization

Fig. 3 Schematic of the proposed methodology

3 Implementation of the procedure

A building located in the city of L'Aquila has been chosen as a case study for implementing the procedure illustrated in the previous sections. The building has an approximate L shape in the plane configuration and five storeys. The structure is made up of RC frames in two directions that are connected by secondary beams. The geometry and the details of the main elements have been derived from the original design drawings.

The floor plan of the building has dimensions of 32.0 m in one direction and 27.0 m in the other, with a total area of about 368 m^2 (Fig. 4). The length of the beams is extremely variable, even within the same frame. The total height of the building is 20 m. It consists of five floors with a storey height of 3.3 m, except for the first floor, which is 3.5 m. The first floor is used as a garage and the other floors for residential purposes.

The overall cast-in situ RC one-way slabs thickness is 24 cm with a deck of about 4 cm which ensure the rigid diaphragm effect for each floor.

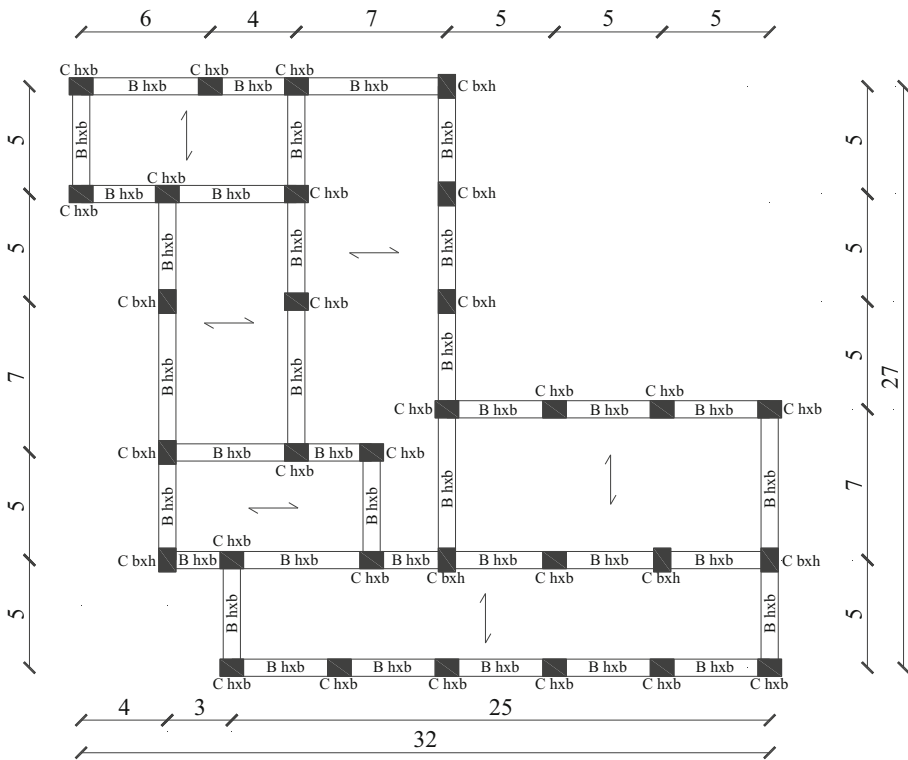Geometrical proprieties of the elements are listed in the Table 1.

Fig. 4 Floor plan of the building (lengths are in meters)

Table 1 Longitudinal and transverse reinforcement details

	Columns	Beams
First storey	0.55 × 0.40, LR: 10Ø16, TR: Ø6/25 cm	0.50 × 0.40, LR: 9Ø12, TR: Ø6/25 cm
Second storey	0.50 × 0.40, LR: 10Ø16, TR: Ø6/25 cm	0.50 × 0.40, LR: 9Ø12, TR: Ø6/25 cm
Third storey	0.50 × 0.35, LR: 8Ø16, TR: Ø6/25 cm	0.50 × 0.35, LR: 7Ø12, TR: Ø6/25 cm
Fourth storey	0.45 × 0.35, LR: 6Ø16, TR: Ø6/25 cm	0.50 × 0.30, LR: 7Ø12, TR: Ø6/25 cm
Fifth storey	0.45 × 0.35, LR: 6Ø16, TR: Ø6/25 cm	0.50 × 0.30, LR: 7Ø12, TR: Ø6/25 cm

LR longitudinal reinforcement, *TR* transverse reinforcement

The longitudinal reinforcement represents the total rebar amount of the beams. The beams reinforcement is not symmetric. Two rebars are in the corners of the compressive zone while the other rebar are located in the tensile zone (equally distanced from each other).

In addition to the original design drawings, several destructive and non-destructive tests were carried out on the building to investigate the material mechanical properties. These tests found that the building consists of structural elements reinforced with smooth bars.

It was possible to determine the following mechanical properties from the destructive and non-destructive tests: the concrete compressive strength $f_{cm} = 12.5$ MPa; and the steel tensile strength $f_{ym} = 279.1$ MPa.

As shown in Sect. 2, the first step of the procedure is the site hazard characterization. The probability of exceeding the intensity of a given earthquake is evaluated as the inverse of the return period. Indeed, in the application case, the vulnerability curve is divided into a discrete number of points that are the eight intervals of observation obtained by the nine return periods of the site.

Nonlinear building response was simulated with finite element software (SAP 2000), using lumped plasticity models of beams and columns (4 hinges for each structural member: top and bottom for both directions). Column and beam plastic hinge models are calculated according to the European Code UNI-EN 1998-3: 2005 (European Standard 2005) as shown in Fig. 5.

In this application case, the EDP is the relative displacement between the various floors, defined as IDR (inter-storey drift). This parameter is the most representative of the structural damage of almost all the components, and is the most simple to assess. All the floor displacements, and then all the relative displacements, are known from the structural analysis. This means that in the application case for the structural and non-structural elements belonging to the same storey, only one EDP, which is the relative inter-storey drift, has been assumed. Once the hazard and EDP have been defined, it is possible to identify the state of the damage to each structural component according to suitable fragility curves. In the application case, if the shear action is higher than the shear strength of an element it has been considered the failure of the element as damage state. Accordingly, in case of shear failure the cost of the damage is equal to the cost of replacement of the RC member. It is then also possible to obtain the economic losses relating to each step of the pushover curve. At this stage, it is necessary to calculate the value of the discount rate in the structure's life-time. The discount rate is largely dependent on two factors, which appear to be closely related: the inflation and interest rates of the central bank. It is very difficult to predict economic performance over a period of several years, and so it is necessary to assume a value of the average discount rate that may realistically occur in the time window. In this application, an annual rate equal to 2% has been assumed.

The nominal life-time of the structure has been chosen to be equal to 50 years, which is the period usually attributed to buildings without any strategic importance. The economic
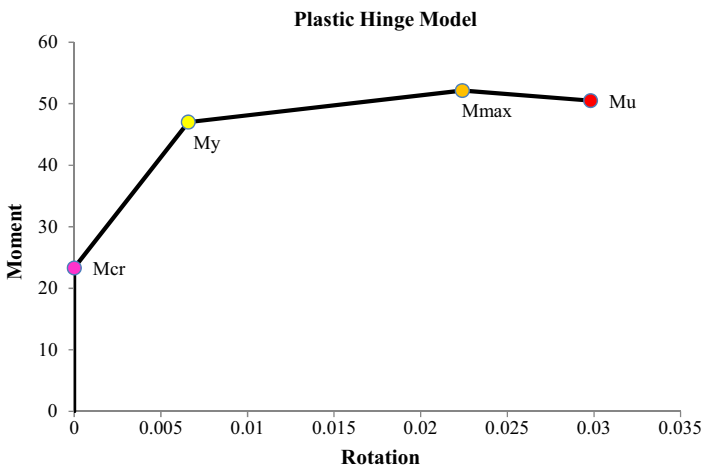

Fig. 5 Plastic hinge model for structural elements

loss of the building can be computed by a simplified equation that multiplies the cost of the economic damage of the pushover step (corresponding to the demand related to several return periods) for the probability of occurrence:

$$L_{V_n} = T_S \times \sum_i \left(C_i \times p_{r,i} \right) \tag{5}$$

where C_i is the cost of a generic step, the subscript i represents the eight time slots considered by the Italian Code, and T_s is the total discount rate.

3.1 Fragility curves

First, it is important to assign an economic value to each component of the building under investigation using a document that allows the components to be associated with a relative economic value. The price list of the Abruzzo region has been used in support of this assessment. This was produced in 2009 after the earthquake of 6 April of that year.

Furthermore, in this application case, the structure is divided into four components (both structural and non-structural): beam-column joints, beams and columns, drywall partition, and systems (i.e. electric system and hydraulic system). Once the building's economic value has been determined, the fragility curves related to each building component have to be assigned in order to predict the potential economic losses for several earthquake scenarios.

3.1.1 Beam-column joints

A study by Pagni and Lowes (2006) was used to define the beam-column joint fragility curves. This defines four damage states (DS):

DS1   First opening of cracks.
DS2   Concrete spalling of at least 30% of the surface of the joint panel.
DS3   Concrete spalling of at least 80% of the surface of the joint panel.
DS4   Collapse of the joint.

Table 2 shows the mean and standard deviation for each DS.
Table 3 provides the repair cost of each DS.

3.1.2 Beams and columns

The estimation of the probability of exceeding a certain level of damage to the beams and columns with a low amount of reinforcement was determined according to the work of Aslani and Miranda (2005). For these elements, the following DS are identified:

Table 2 Joint fragility curves (Pagni and Lowes 2006)

Joints	IDR	
	Mean (%)	SD
DS1	1.40	0.57
DS2	2.60	0.50
DS3	3.10	0.45
DS4	3.70	0.26

Table 3 Repair cost of each damage state

Joints

Damage state	Repair efforts	Unit cost (€/m^2)
DS1	House painting	15
DS2	Cleaning of the concrete surfaces	10
	Resin injection	179
DS3	Removing of the damaged concrete surfaces	10
	Cleaning of the concrete surfaces	10
	Resin injection	179
	Adjustment of steel reinforcement	131
DS4	Removing of the damaged concrete surfaces	10
	Cleaning of the concrete surfaces	10
	Resin injection	179
	Adjustment of steel reinforcement	131
	Re-arrangement of the steel bars	50

DS1   light cracking.
DS2   severe cracking.
DS3   member shear failure.

The mean and standard deviation values depend on the geometrical properties of the elements. For this reason, fragility curves have been calculated for the elements belonging to each floor of the building under investigation in the present study. Table 4 shows the mean and standard deviation values obtained for the structural elements of the first floor.

Table 5 provides the repair cost of each DS.

Table 4 Beam and column fragility curves (Aslani and Miranda 2005)

Beams and columns	IDR	
	Mean (%)	SD
DS1	0.35	0.37
DS2	0.75	0.44
DS3	1.00	0.58

Table 5 Repair cost of each damage state

Beams and columns

Damage state	Repair efforts	Unit costs
DS1	House painting	15 €/m^2
DS2	Cleaning of the concrete surfaces	10 €/m^2
	Resin injection	179 €/m^2
DS3	Cleaning of the concrete surfaces	10 €/m^2
	Resin injection	179 €/m^2
	Adjustment of steel reinforcement	131 €/m^2
	Re-arrangement of the steel bars	50 €/n°

3.1.3 Drywall partitions

A study by Ruiz-Garcia and Negrete ([2009](#)) was used to define fragility curves related to internal and external partitions. This contains a database of experimental tests carried out on various types of partition element, some of them compatible with Italian ones. For the definition of fragility curves for drywall partitions, it is common to only use two DS:

DS1   formation of cracks on the member surface no larger than 0.1 mm.
DS2   formation of X-shaped cracks on the member surface of about 5 mm and relevant concrete spalling in the beam-column joint panel.

The parameters related to the fragility curves are shown in Table 6.
Table 7 provides the repair cost of each DS.

3.1.4 Systems

In implementing the procedure, it has been assumed that the systems need to be replaced if they are within very damaged partitions (i.e. the partition has to be demolished). Accordingly, the fragility curve of their only DS is perfectly equal to DS2 of the drywall partitions. This means that if the partitions achieve DS1 as the damage state, the systems do not need to be replaced.

3.2 Economic loss

The economic loss is given as the sum of the repair costs of the damaged components at each ground-motion IM (i.e. PGA or d_r) and the cost related to the unavailability time of the facility, named the cost of building unavailability in the following.

3.2.1 Component repair costs

Each DS corresponds with one or more repair processes. The sum of the repair processes' costs provides the actual economic loss associated with a component (structural or non-structural). The economic loss is expressed as the ratio between the repair and

Table 6 Partition fragility curves (Ruiz-García and Negrete [2009](#))

Partitions	IDR	
	Mean (%)	SD
DS1	0.10	0.73
DS2	0.35	0.57

Table 7 Repair cost of each damage state

Partitions		
Damage state	Repair efforts	Unit costs
DS1	House painting	15 €/m^2
	Plaster	25 €/m^2
DS2	Demolition	80 €/m^3
	Reconstruction	501 €/m^3

reconstruction costs of the component. It is worth nothing that, for a severe DS, the cost of repair could largely exceed the reconstruction cost (i.e. the economic loss in this case is greater than 1).

3.2.2 Casualties and injuries costs

The framework proposed could be enriched with also losses related to injuries and casualties as a number of references can be used to quantify the cost of human life (e.g. Coburn and Spence 2002). Introduction of costs related to human life could increase the benefit/cost ratios in some cases up to 8 times, thus shifting the outcome of the analysis towards the feasibility of retrofit (Kappos and Dimitrakopoulos 2008). Nevertheless, this aspect is out of the scope of the application case even if the framework could be enriched to include it.

3.2.3 Cost of building unavailability

In order to determine the economic losses, it is also necessary to evaluate the costs related to the unavailability of the building due to a destructive earthquake. In particular, in the case of seismic actions that produce a certain level of structural damage, the building may not be usable. As a consequence, additional costs should be computed by accounting for the payment of alternative accommodation for those who lived in the building. This sum, of course, depends on how long the building is unavailable. The unavailability cost for each person has been evaluated in the present application taking into account the fact that each inhabitant of the building must be hosted in a hotel for the entire period the building is unavailable. In the application case, the average daily cost of staying in a hotel was estimated to be about €17.00 per person. According to National Statistics Institute (ISTAT) data (http://www.tuttitalia.it/abruzzo/provincia-dell-aquila/statistiche/popolazione-andamento-demografico/), in L'Aquila there is an average density of three persons per dwelling. Accordingly, in total, the daily cost of unavailability in the case study has been computed as follows:

$$C_{in} = n_{ap} \times d_{ab} \times C_{pers} \tag{6}$$

where n_{ap} is the number of dwellings in the building, d_{ab} is the average density for each dwelling, and C_{pers} is the daily cost of a hotel stay for each resident. In this application case, n_{ap} is 8 and thus the daily total cost of the unavailability of the building is approximately €408.00.

The usability disruption is very complicated to predict, due the variability of many factors. In this study, an unavailability time as a function of the level of the structural damage has been established. This time has been assumed to be in a range between six and 18 months, and has been evaluated as the ratio between the loss due to structural damage and the cost of unavailability for six months. This ratio is assumed to be at least one and no more than three. For partial or total collapse, or for very severe structural damage (i.e. if demolition is needed), an unavailability time of 36 months has been assumed.

So, at this stage, the economic loss L_{Vn} of the building over its structural life-time can be computed according to Eq. 5. In this study, Eq. 5 provides the following loss in Euros:

$$L_{V_n} = T_S \times \begin{bmatrix} C_{30-50} \times p_{r,30-50} + C_{50-72} \times p_{r,50-72} + C_{72-101} \times p_{r,72-101} + C_{101-140} \times \\ p_{r,101-140} + \quad C_{140-201} \times p_{r,140-201} + C_{201-475} \times p_{r,201-475} + C_{475-975} \times \\ p_{r,475-975} + C_{975-2475} \times p_{r,975-2475} \end{bmatrix}$$

$$\approx 1350k\text{€}$$

This economic loss corresponds to the original building structural capacity (i.e. the safety level of the building if no strengthening interventions are made). If the capacity of the building needs to be increased, as commonly happens in existing structures, several strengthening strategies and relevant techniques can be selected. Each strategy implies an intervention cost as a function of the target safety level. In the present study, several strengthening techniques have been investigated and relevant costs have been determined in order to define the total expected cost curves. According to these curves, it is possible to select the strengthening strategy that minimizes the total expected costs with a maximum safety security level.

4 Strengthening intervention strategies

Strengthening strategies aiming at increasing ductility, stiffness, and strength, or all of them, have been selected, as is common practice. In particular, according to these goals, the following strengthening techniques have been investigated in this study:

1. FRP-based strengthening solution (i.e. shear strengthening of beam-column joints, columns, and beams using FRP sheets to prevent brittle failure mechanisms, and the confinement of columns at the ends by means of FRP wrapping to increase the structural global ductility).
2. RC jacketing-based strengthening solution (i.e. RC jacketing of beams and columns to increase the flexural and shear capacity of members, as well as ductility, and to increase the global structural stiffness).
3. FRP–RC jacketing-based strengthening solution (i.e. a combined strengthening solution based on both of the previous solutions according to the main deficiencies of the structural members. This allows a slight increase in the building's global stiffness to be balanced with the local increase in shear capacity to prevent brittle failure mechanisms).
4. Insertion of RC shear wall-based strengthening solution (i.e. insertion of two shear walls to sustain the seismic action in both the longitudinal and transverse directions).
5. Base isolation-based strengthening solution (i.e. inserting a horizontally flexible and dissipative interface on the first floor of the building, thus significantly reducing the demand rather than increasing the structural capacity, as in the previous listed strengthening solutions).

The first method consists of the application of one or more FRP sheets to the surface of the beam-column joint panels and on beams and columns as shear strengthening. The second and third intervention strategies aim to improve the seismic performance of the individual elements with RC jacketing with a thickness of 5 cm or the application of FRP fabrics as described above against shear failures. With these intervention strategies, the structure increases its capacity in terms of both stiffness and ductility. The fourth strategy aims to increase the stiffness of the structure by the insertion (compatibly with the

geometry of the structure) of two RC shear walls for both directions. The insertion of the shear walls does not, however, enable all the brittle crises of joints, beams, and columns to cope with (which is also necessary) the application of FRP fabrics.

The fifth strategy consists of the insertion of rubber bearings and friction isolators between the first and second floors. The structure rests on these devices, which allow significant relative displacements and provide sufficient energy dissipation in order to limit them. In this way, the building's movement is decoupled from the soil movement, producing an increase in the structural vibration period. In the isolation strategy, the building must achieve a target period which corresponds to the target spectral acceleration in the inelastic spectra demand. The target spectral acceleration is a function of the step of the pushover curve where the first brittle failure occurs. However, the insertion of the isolation devices does not allow all the brittle joints, beams, and columns to cope with (which is necessary) the application of FRP sheets.

In order to carry out an analysis of the economic viability of a strengthening strategy, it has been assumed that the performance of the building at different strengthening levels is improved. The strengthening levels have been related to the safety levels, which are computed as the ratios between the structural capacity and the seismic demand in terms of the PGA. The safety level of 100% corresponds to strengthening interventions providing a structural capacity equal to the structural demand related to a severe earthquake with a return period of 475 years (i.e. the safety level currently required for new ordinary buildings designed according to current seismic code provisions).

Two non-linear static analyses have been performed for the two plan directions of the structure independent from each other (x–x and y–y directions). Accordingly, the most unfavourable from an economic point of view has been chosen (y direction). The horizontal load pattern assumed in the analysis is a first mode force pattern and has been defined according to the European Building code (European Standard 2004). This horizontal load pattern is obtained from the displacement distribution of the modal analysis. The pushover curve has been divided into different points, and each of them corresponds to a safety level. For each safety level, a structural analysis has been performed to identify all the brittle failures (shear failure on beams, columns, or beam-column joint panels). This allows us to determine a list of elements that needs to be strengthened (i.e. capacity lower than the demand). A price is associated with each action necessary for the strengthening of the element, the sum of the cost of the materials, and the manual workers required. Note that if the strengthening intervention modifies the structural stiffness, the pushover curve has to again be determined at each step of the analysis (i.e. the effective structural period changes and so does the displacement demand).

With the progress of the pushover curve (i.e. by increasing the top displacement), there is an increase in the failures that may occur in the elements. Increasing the number of failures, obviously, also increases the cost of achieving a given safety level for the structure.

The result is a cost curve that gradually increases with the increase of the safety level of the building, as shown in Fig. 6 for each selected strengthening strategy. The curves have been computed up to a safety level of 100%. Table 8 provides also a breakdown of prices of each retrofit scheme.

In Fig. 6, the dashed line represents the cost trends, which have been determined only for selected points. The safety increase may imply one or more strengthening interventions on different structural members depending on the retrofit strategy and technique; in the case of FRP based strategy it is possible a selective strengthening strategy, the costs gradually increase by slightly increasing the structural safety level. For each failure
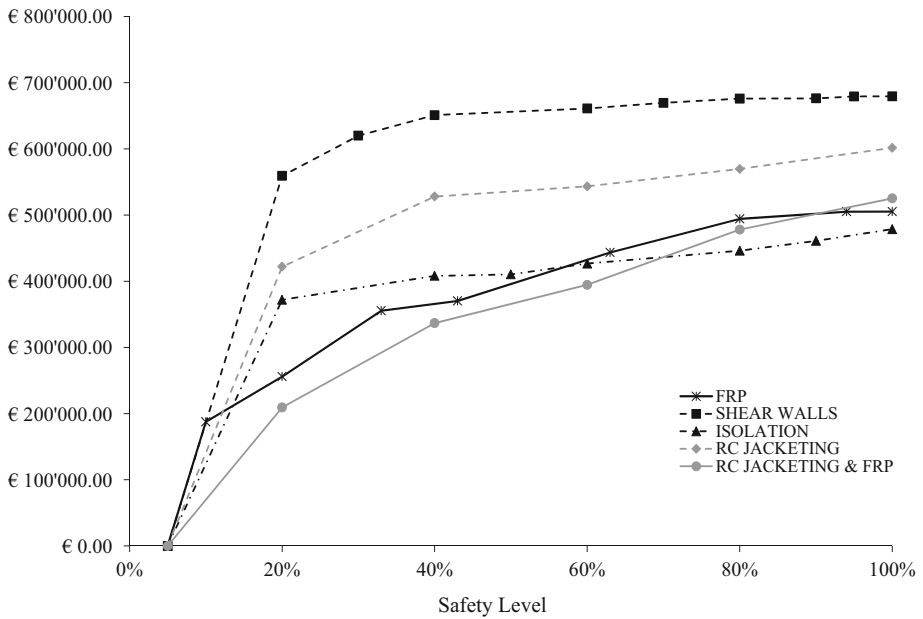
Fig. 6 Cost of the strengthening interventions

Table 8 Breakdown of prices of each retrofit solution

Strengthening strategies	Unit costs
FRP[a]	370 €/m^2/(n° of layer)
SHEAR WALLS[b]	3830 €/m^3
BASE ISOLATION[c]	9822 €/(n° of device)
RC JACKETING[d]	9960 €/m^3
RC JACKETING and FRP[e]	9960 €/m^3

The cost of strengthening works have been obtained from the price list of the Abruzzo region (Anno 2009)

For RC Jacketing and RC Jacketing and FRP the unit costs are average values, because the influence of demolition and reconstruction of non-structural elements on the unit costs depend on the strengthening target that one wants to achieve. The unit costs reported are evaluated considering as unit measure the amount of concrete casting necessary for the strengthening of the RC elements

[a] Crack injections, sand blasting, primer, putty, saturant, demolition and reconstruction of partitions and partition paintings are included

[b] Rebars arrangement, formwork, concrete casting, foundation strengthening, demolition and reconstruction of partitions, partition paintings and check or restoration of all the systems (water supply, electric installation, etc.)

[c] Retrofit procedures, installation and maintenance of devices, execution tests and steel plate for the foundations are included

[d] Rebars arrangement, formwork, concrete casting, demolition and reconstruction of partitions, partition paintings and check or restoration of all the systems (water supply, electric installation, etc.)

[e] All the operations computed for a and d are included

corresponding to a given safety level, a localized strengthening solution may be designed with a relevant slight cost increase; this is possible because FRP does not imply stiffness variation. Accordingly, the curve may be obtained by connecting several points corresponding to different safety levels and strengthening costs. The curve related to the FRP-based strategy has an almost linear trend, except for the first branch. A similar trend can be also observed on the curve related to the FRP and/or RC jacketing strengthening strategy.

In the other cases (i.e. shear walls, base isolators, and RC jacketing), the curves show an initial strong increase in costs, even for a slight increase of safety levels. This because the stiffness or structural period is significantly changed in order to improve the structural seismic capacity or reduce the seismic demand. Then, with low additional costs, the safety level can be significantly increased by up to 100% (i.e. the curve has an almost constant trend). This is because these strategies imply a significant initial cost investment of applying the strengthening technique (e.g. the insertion of shear walls on each floor or the insertion of base isolators at the foundation level are clearly costly interventions), but then only a few members may need to still be strengthened to avoid localized failures. Table 9 shows the number of structural elements strengthened for each safety level.

Table 9 Number of structural elements strengthened for each safety level

	Safety level				
	20%	40%	60%	80%	100%
FRP	FSS to 44 columns and 37 beams	FSS to 3 beam–column joints, 84 columns, 42 beams	FSS to 303 beam–column joints, 105 columns, 50 beams	FSS to 433 beam–column joints, 118 columns, 59 beams	FSS to 483 beam–column joints, 124 columns, 68 beams
Shear walls	RCSW; FSS to 633 beam–column joints, 31 columns, 39 beams	RCSW; FSS to 643 beam–column joints, 92 columns, 39 beams	RCSW; FSS to 653 beam–column joints, 94 columns, 53 beams	RCSW; FSS to 673 beam–column joints, 104 columns, 66 beams	RCSW; FSS to 683 beam–column joints, 105 columns, 75 beams
Base isolation	BID; FSS to 2 columns	BID; FSS to 6 columns	BID; FSS to 10 columns	BID; FSS to 43 beam–column joints, 11 columns, 8 beams	BID; FSS to 123 beam–column joints, 11 columns, 17 beams
RC jacketing	RCJ to 131 columns, 84 beams	RCJ to 165 columns, 88 beams	FSS to 53 beam–column joints; RCJ = 165 columns, 152 beams	FSS to 313 beam–column joints; RCJ to 165 columns, 162 beams	FSS to 613 beam–column joints; RCJ to 165 columns, 166 beams
FRP and RC jacketing	FSS to 57 columns, 49 beams; RCJ to 35 columns	FSS to 80 columns, 61 beams; RCJ to 70 columns	FSS to 68 columns, 77 beams; RCJ to 95 columns	FSS to 93 beam–column joints, 25 columns, 104 beams; RCJ to 140 columns	FSS to 113 beam–column joints, 114 beams; RCJ to 165 columns

FSS FRP shear strengthening, *RCSW* 4 RC shear walls (2 per direction), *BID* base isolation devices, *RCJ* RC jacketing
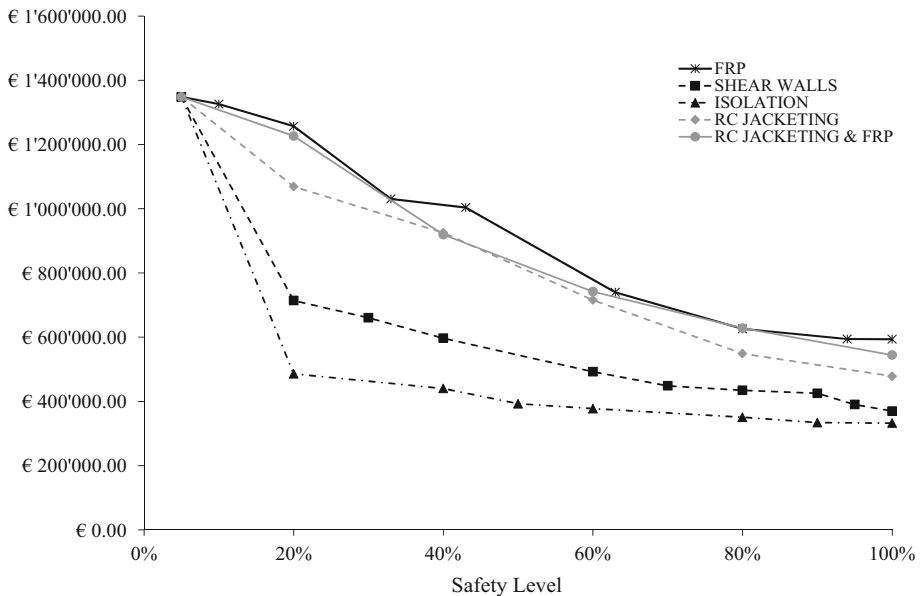
Fig. 7 Expected economic losses

The next step consists of calculating the economic loss of the structure, according to the procedure described above. This assessment is made for the different safety levels for which the building is gradually strengthened. The economic loss trend related to each strengthening strategy is depicted in Fig. 7. As expected, the curve trend is again almost linear for the FRP and the FRP and/or RC jacketing strategies, while an initial significant loss reduction is shown for the other strategies.

To check if a reinforcement intervention is cost effective for an owner, it is necessary to add the cost of the strengthening intervention and the loss of the structure for each safety level, thereby obtaining the total expected cost. This graph is shown in Fig. 8. The graph shows that, for the case under investigation, the isolation strategy is the most cost-effective solution. The curve has a decreasing trend up to the optimal point, which corresponds to a safety level of 90%, with an expected total loss reduction of about 40% (i.e. 810,000€/ 1,350,000€) with respect to the no strengthening intervention case (for which the safety level is about 5%). In the other cases, the optimal point corresponds to 100% of the safety level, with an expected total loss reduction in the range of 28–32%. The strategy based on the insertion of RC walls also shows a strictly decreasing trend. The difference between these strategies and the other three is that the curves related to them have an initial increasing, and then a decreasing, trend. This means that, in order to define the most cost-effective intervention, in the case of the FRP-based or FRP and/or RC jacketing-based strategies, at least a certain minimum safety level should be attained to reduce the total expected losses with respect to the case of no strengthening: almost 40% for FRP combined with RC jacketing; and 50 and 55% for the FRP and RC jacketing strategies, respectively. If such safety levels are not attained, the strengthening solution, although it provides a benefit in terms of safety, is not economically viable. This confirms that the selection of the most effective strengthening strategy from both a structural and economical point of view is a challenging task. Furthermore, each strengthening strategy may
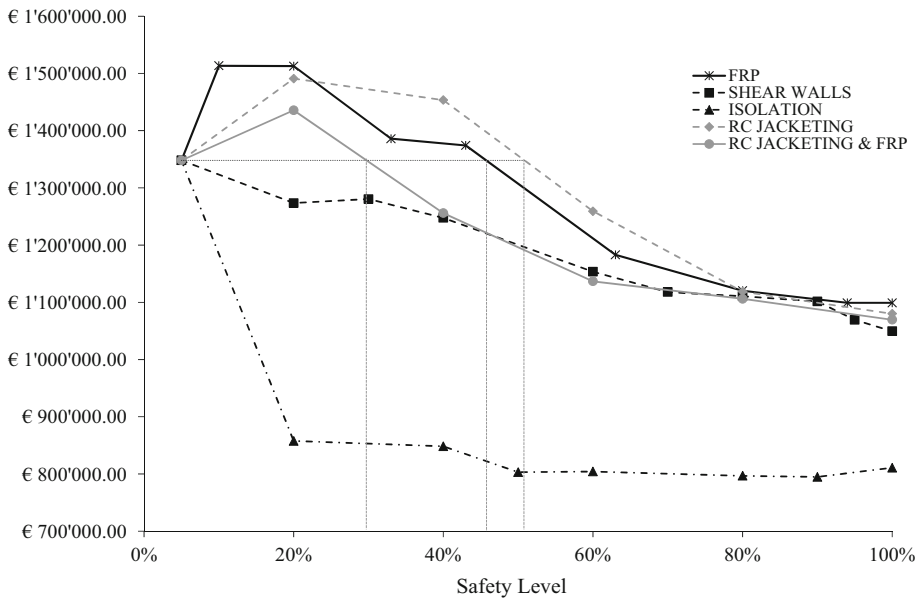
Fig. 8 Total expected costs

imply a different minimum safety level to reduce losses in the structural life-time of the building.

Finally, it may be interesting to underline that the building chosen as case of study was severely damaged by the 2009 L'Aquila earthquake. Therefore, according to practitioners calculation the repair and strengthening interventions were economically not viable. Therefore, the building was demolished and rebuilt with a total amount of 2,000,000 Euros which is significantly higher than optimal expected total cost.

5 Multi-hazard analysis

The results of a life-cycle cost analysis, as shown, depend on different parameters. In fact, it is clear that the final results will be different when applying the same procedure, with the same fragility curves and the same strengthening strategies, to different structures or different building locations. For this reason, a multi-hazard analysis has carried out to investigate the influence of the local seismic hazard on the most cost-effective solution. In the analysis, it is assumed that the building previously investigated is located in different sites with different PGA values belonging to four different seismic zones. In particular:

- Zone 1—High seismicity [PGA higher than 0.25 g] (which is the case previously analyzed).
- Zone 2—Mean seismicity [PGA between 0.15 and 0.25 g].
- Zone 3—Low seismicity [PGA between 0.05 and 0.15 g].
- Zone 4—Very low seismicity [PGA lower than 0.05 g].

The value of the economic loss of the building gradually decreases with the decreasing intensity of the PGA. Indeed, the probability of occurrence of the eight earthquakes is the
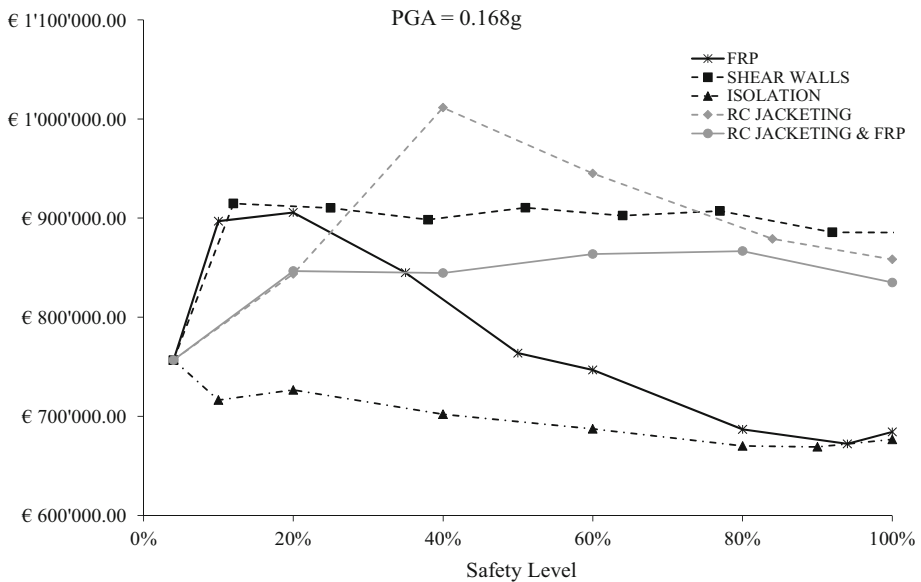
Fig. 9 Total expected costs for PGA = 0.168 g

same, but in each case the damage to the components changes. In Figs. 9, 10, and 11, the total expected loss curves are reported with reference to the mean, low, and very low seismicity zones (i.e. a PGA demand corresponding to a return period of 475 years has been assumed to be equal to 0.168, 0.071, and 0.049 g).

The figures show that the isolation strategy is also the most cost-effective strengthening intervention for a PGA value belonging to the mean seismicity zone (see Fig. 9). In this case, the FRP strategy is very competitive, but a safety level of at least 50% has to be attained in order to define the most economically advantageous intervention. The other three strategies are definitely not effective from an economic point of view, because the total expected costs are greater than those related to the case of a no strengthening intervention in the useful life-time of the structure, L_{Vn}.

In the low and very low seismicity zones, the isolation strategy cannot be applied to the structure under investigation. In fact, the target period to achieve with this solution is lower than the fundamental period of the structure. This is not compatible with the concept of base isolation, in which the structural period of the vibration increases.

In these seismicity zones, Figs. 10 and 11 show that the FRP-based strategy is the best and the only cost-effective strengthening intervention strategy. The optimal point is at a safety level of 100 and 60% for low and very low seismicity, respectively. The corresponding expected total loss reductions with respect to the case of the no strengthening intervention are about 84 and 75%, respectively. The other strategies are not economically viable.

The application case clearly shows that the selection of the most effective strengthening strategy from a structural and economic point of view strongly depends on the hazard posed by the area where the building is located.
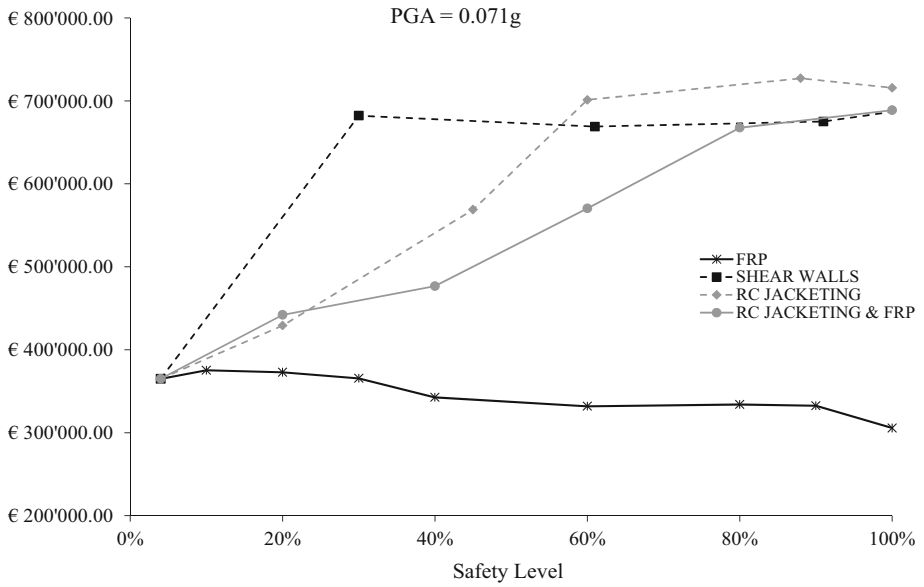
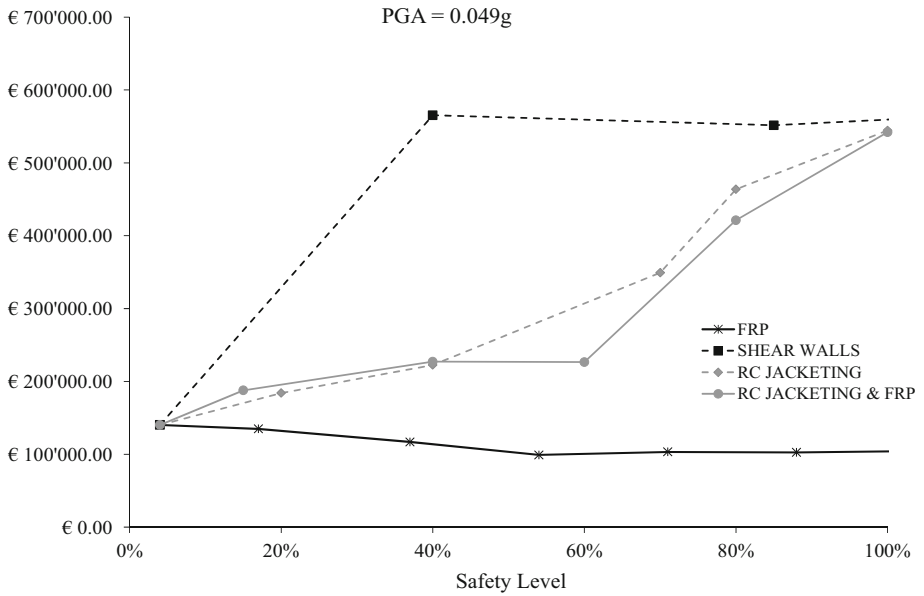**Fig. 10** Total expected costs for PGA = 0.071 g

Fig. 11 Total expected costs for PGA = 0.049 g

6 Conclusions

In this study, a semi-probabilistic methodology is proposed for assessing the economic performance of a building prone to seismic risk. The methodology is based on the PEER's approach by replacing the use of non-linear time-history structural analysis by means of a static non-linear one. In particular, the proposed methodology is based on the use of a non-linear static analysis carried out excluding the torsional effects, the occurrence of plastic hinges due to shear deformation, and assuming only one EDP for the structural and non-structural elements belonging to the same storey, as commonly assumed by practitioners involved in the assessment of seismic capacity of existing buildings. In order to investigate whether this simplified approach is promising for common applications, a single example building has been analysed. The methodology aims providing a tool to:

- Assess economic losses for existing and new buildings.
- Evaluate different retrofit scenarios of existing buildings.
- Optimize the seismic strengthening of existing structures.

It was possible to obtain three different kinds of total expected cost versus safety level curve. In the first case, the curve presents a decreasing and then an increasing branch. In this case, the most-cost effective solution is simply identified by the lowest value of the curve. In the second case, there is an increasing branch followed by a decreasing branch. In this case, it is necessary to almost achieve a certain safety level to have expected costs that are lower than the economic losses. In the third case, the curve steadily increases with the safety levels, and so the best solution depends on the target safety level. In each case, the optimal choice of the strengthening intervention should be taken as a balance between the reduction of expected seismic loss in the structural safety life-time and a proper safety level selected according to social factors.

In the application of the procedure, the availability of the fragility curves is a critical issue to consider. Fragility curves specifically targeted at existing buildings in the Mediterranean area are still lacking and need to be determined.

Finally, the analyses show that the most cost-effective intervention strategy may strongly depend on the hazard posed by a building's location. In the case presented in this paper, the base isolation resulted in the most effective strengthening solution for the high PGA values, while the FRP-based strengthening solution was the most effective option for a lower seismic area.

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