

Article

Expected Annual Loss as a Global Metric for Seismic Performance Assessment of Existing Buildings

Roberto Nascimbene ^{1,2,*} and Emanuele Brunesi ²¹ Department STS, IUSS—Scuola Universitaria Superiore Pavia, 27100 Pavia, Italy² EUCENTRE—European Centre for Training and Research in Earthquake Engineering, 27100 Pavia, Italy; emanuele.brunesi@eucentre.it

* Correspondence: roberto.nascimbene@iusspavia.it or roberto.nascimbene@eucentre.it

Abstract

The assessment of seismic performance of existing buildings has traditionally focused on structural safety and damage limitation, often neglecting the explicit quantification of the associated economic consequences. In recent years, performance-based earthquake engineering (PBEE) frameworks have enabled a direct link between structural response and probabilistic loss estimation, allowing economic metrics to be integrated into seismic risk evaluation. Among these, the Expected Annual Loss (EAL) represents a comprehensive indicator that accounts for seismic hazard, structural vulnerability, and exposure over the building's lifetime. This study presents a performance-based seismic loss assessment of an existing reinforced concrete building, adopting EAL as a global metric for seismic performance evaluation. The case study concerns an existing hospital building designed primarily for gravity loads and representative of a large portion of the Italian building stock. A detailed nonlinear numerical model is developed using OpenSees ver. 3.8.0, incorporating shear-critical behavior through nonlinear link elements. Structural performance is evaluated through modal analysis, pushover analysis, and nonlinear time-history analyses using a set of ground motions selected and scaled according to intensity-based criteria. Seismic losses are estimated following the FEMA P-58 methodology, implemented through the PACT software ver. 3.1.2, integrating structural response demands, component fragility functions, collapse probability, and seismic hazard curves. Probabilistic loss curves are derived, and the EAL is computed as a synthetic indicator of economic seismic performance. The results highlight the effectiveness of EAL in capturing the combined effects of seismic hazard and structural vulnerability, demonstrating its potential as a robust decision-support metric for seismic risk mitigation, retrofit prioritization, and insurance-related applications for existing buildings.

Keywords: existing buildings; performance-based earthquake engineering; seismic risk evaluation; Expected Annual Loss (EAL); reinforced concrete building; FEMA P-58

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1. Introduction

Natural hazards continue to pose a major threat to societies worldwide, generating substantial human and economic losses each year. Among them, earthquakes represent one of the most severe sources of risk due to their destructive potential and wide-ranging consequences on the built environment. At the global scale, seismic risk translates into

significant economic burdens, often quantified as a percentage of the gross domestic product (GDP) [1–5], affecting both developed and developing countries [6–8]. Europe is not exempt from this condition, and within the European context, Italy stands out as one of the most seismically exposed countries, ranking among the highest in terms of potential economic losses relative to national wealth [9–13].

Seismic events can lead to catastrophic scenarios characterized by casualties, disruption of essential services, and extensive damage to residential and strategic assets. Historically, a significant portion of the economic losses has been compensated through public funding mechanisms, while the role of private insurance systems has remained relatively limited [14–16]. This imbalance exposes national economies to considerable financial pressure and highlights the need for more resilient and sustainable risk-sharing strategies, including a broader adoption of insurance-based solutions specifically designed to address natural hazards [17–19].

The magnitude of seismic risk is further emphasized when considering the exposure of the building stock. For instance, in Italy alone, millions of residential units are located in areas classified as medium to high seismic hazard, representing a total asset value of several trillion euros [11,20–23]. This risk is amplified by the strong concentration of household wealth in real estate and by the high percentage of homeownership, which makes families particularly vulnerable to seismic losses. Despite this, insurance penetration for natural hazards remains extremely low, covering only a marginal fraction of the building stock [23–25]. Similar trends, although with regional differences, can also be observed in other European countries, reinforcing the need for improved risk quantification and management tools at both national and international levels.

Post-earthquake reconnaissance studies [26] have repeatedly highlighted recurring patterns of structural damage across different seismic events [27,28]. These observations reveal significant vulnerabilities affecting a wide range of construction typologies, including reinforced concrete buildings [27,29–31], precast systems [27,32–34], lightweight concrete structures with infill panels [35–37], and steel frameworks [27,38–40]. Such deficiencies are largely associated with the fact that a substantial portion of the existing building stock was designed according to outdated seismic provisions [41,42], which did not incorporate modern performance-based criteria. As a result, many structures exhibit inadequate seismic behavior when subjected to current hazard levels.

Recent earthquakes have also shown that even moderate seismic events can produce non-negligible damage, not only to structural components but also to non-structural elements [43–46]. The associated repair costs required to restore pre-event conditions can represent a significant share of the total reconstruction cost and, in some cases, may even become economically unsustainable, particularly in regions characterized by moderate-to-high seismicity. These findings have progressively shifted the focus of seismic engineering from a life-safety-oriented perspective toward a broader evaluation framework that explicitly considers economic consequences and functionality losses [47–52]. Within this context, the present study focuses on the seismic performance assessment of an existing reinforced concrete building constructed in the 1970s, representative of a large portion of the European building stock designed primarily for gravity loads [27]. The selected case study is a hospital facility, thus characterized by strategic importance and the need to ensure post-earthquake functionality. The building exhibits construction details, material properties, and design assumptions that differ significantly from those adopted in modern practice, reflecting the substantial evolution of seismic design codes over the past decades [27,41,42,53].

The analysis is conducted within the framework of Performance-Based Earthquake Engineering (PBEE), as formalized in the FEMA P-58 [54,55] methodology developed by the Federal Emergency Management Agency in collaboration with the Pacific Earthquake

Engineering Research (PEER) Center and other research institutions. This approach provides a probabilistic framework that integrates seismic hazard, structural response, and damage assessment, ultimately leading to quantitative estimates of performance in terms of economic losses [50,55–57]. The methodology relies on the definition of fragility functions for both structural [58] and non-structural components [59], as well as on the evaluation of repair costs, downtime, and potential casualties associated with different damage states.

The implementation of the PBEE methodology in this study is carried out using the PACT ver. 3.1.2 (Performance Assessment Calculation Tool) software [60–62], which enables a comprehensive loss assessment by combining structural analysis results with probabilistic models of damage and consequence. The building model incorporates information on geometry, construction costs, and occupancy patterns, together with engineering demand parameters derived from nonlinear analyses.

The primary objective of this research is to identify and evaluate a synthetic performance indicator capable of quantifying the expected economic losses associated with seismic events over time. In this regard, the Expected Annual Loss (EAL) is adopted as a global metric, as it explicitly accounts for both the seismic hazard and the vulnerability of the structure, providing a direct measure of the average yearly economic impact [63,64]. The use of EAL allows for a more comprehensive and decision-oriented assessment of seismic performance, providing a quantitative economic metric that may support future risk-informed decision-making and retrofit evaluation processes.

2. Structural Description and Numerical Model

The case study building examined in this research belongs to a larger hospital complex composed of several structurally distinct blocks (Figure 1a—Policlinico of Modena, city of Modena, Italy). Owing to the presence of structural joints between adjacent units, the seismic assessment could be limited to a single building (Block A) without explicitly considering the interaction with the surrounding parts of the complex. According to the current urban planning classification, the area is designated for healthcare and hospital facilities, while the specific block investigated in this study is intended for inpatient use. This aspect is also relevant for the subsequent economic assessment, since the building function affects the estimation of replacement and repair costs.

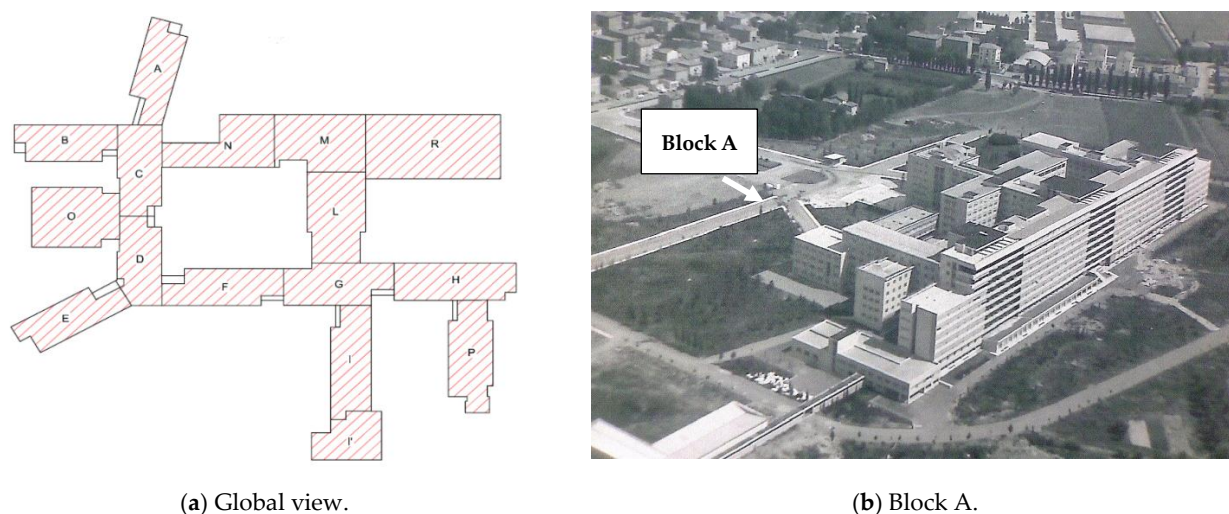
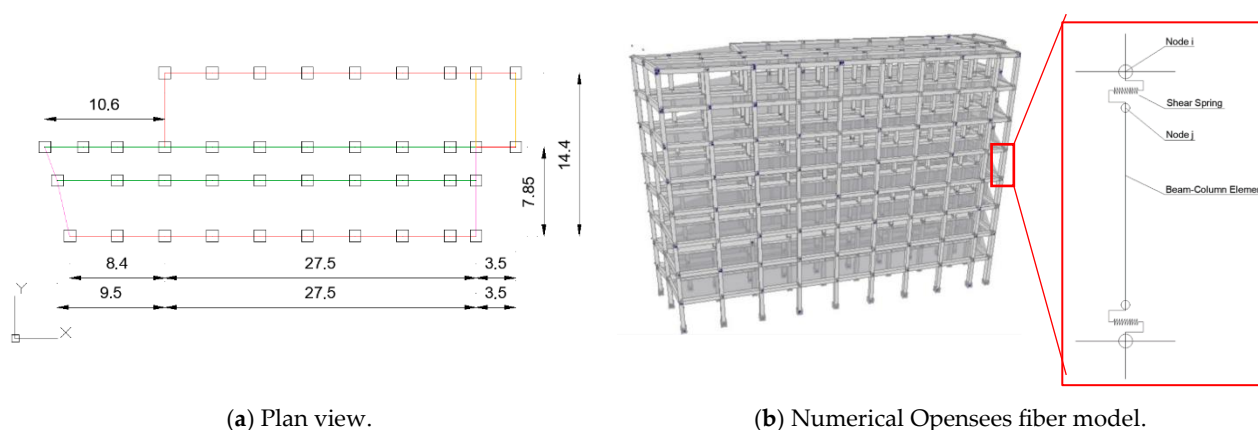


Figure 1. (a) Overview of the hospital complex, highlighting the different structural blocks; (b) view of the analyzed building (Block A), selected for the seismic and loss assessment.

Although the selected case study is located in Italy, the investigated structural typology is representative of a much broader class of existing reinforced concrete buildings constructed before the widespread adoption of modern seismic design principles. Similar gravity-load-designed buildings can be found throughout Southern Europe, the Mediterranean region, and many seismic areas worldwide. Consequently, the significance of the present study extends beyond the specific case considered, providing insights into the expected seismic losses and long-term economic consequences associated with a large stock of vulnerable existing buildings.

The structure consists of a reinforced concrete moment-resisting frame system (Figure 1b) with a total height of 33.8 m, including 27.0 m above ground and 6.8 m below ground. The plan layout, shown in Figure 2a, remains essentially constant along the building height and refers to a ten-level configuration. From a structural standpoint, the building is composed of four main frames parallel to the x -direction, spaced in the transverse y -direction by 4.90 m, 2.95 m, and 6.55 m, respectively. These frames are mainly connected by perimeter (secondary) beams, with the exception of one specific beam element highlighted in Figure 2a (in yellow color).

In elevation, the building comprises eight above-ground stories and two basement levels. For the purposes of the structural model developed in this study, the eight above-ground stories were explicitly considered as the primary seismic-resisting portion of the structure (Figure 2b). The typical interstory height is 3.40 m, except for the ground floor, where it is equal to 3.20 m. The vertical resisting system is made of columns with cross-sections increasing toward the foundation level, consistently with gravity load demand and tributary area. More specifically, the column dimensions vary from a minimum cross-section of 0.40 m $\times$ 0.40 m at the upper stories, where 41 columns are present at the seventh floor, up to a maximum cross-section of 0.55 m $\times$ 0.55 m at the foundation level. All columns are characterized by a concrete cover of 20 mm and transverse reinforcement consisting of two-legged stirrups with diameter 6 mm spaced at 25 cm. The beam system also varies depending on the position in plan, although the beam arrangement is repeated story by story. Internal beams have a cross-section of 1.20 m $\times$ 0.30 m, whereas perimeter beams, arranged in an edge configuration, exhibit cross-sections equal to 0.30 m $\times$ 0.60 m, 0.50 m $\times$ 0.40 m, and 0.40 m $\times$ 0.60 m.



(a) Plan view.

(b) Numerical OpenSees fiber model.

Figure 2. (a) Plan view of the analyzed building (Block A), showing the structural layout and frame distribution; (b) three-dimensional OpenSees numerical model based on fiber-section elements used for nonlinear analysis.

Once the main geometric and mechanical features of the building had been identified from the original design drawings, a numerical model was developed in OpenSees [65] (Figure 2b). The analytical model was initially constructed using the classical distributed-plasticity fiber-section formulation originally proposed by Spacone et al. in [66] and

widely adopted in nonlinear seismic analysis of reinforced concrete frame structures (*section Fiber* as OpenSees command). This modeling strategy is well established for capturing flexural nonlinearities and the spread of plasticity along beams and columns. However, for existing reinforced concrete buildings designed prior to modern seismic provisions [27], shear behavior may become a critical issue [67,68], especially in elements with inadequate transverse reinforcement and limited ductility capacity. In such cases, a flexure-dominated fiber model alone may not be sufficient to represent the actual response, as it can underestimate brittle or semi-brittle shear-related damage mechanisms [69–71]. For this reason, the original OpenSees model [65,66] was enhanced in order to improve the representation of the shear response at the ends of beams and columns. Rather than assuming full continuity of the elastic frame element at its extremities, a concentrated nonlinear shear link was introduced through a zero-length (*ZeroLength* as OpenSees Command) element associated with an appropriate constitutive law (*UniaxialMaterial Pinching4* as OpenSees command). This modeling approach made it possible to decouple the flexural response of the member from its local shear behavior and to explicitly simulate the progressive degradation of shear resistance. The final modeling layout is shown schematically in Figure 2b related to the homemade shear improvement of the originally implemented in OpenSees fiber model.

The nonlinear shear springs were implemented in OpenSees through the *UniaxialMaterial Pinching4* command. Starting from the basic hysteretic formulation permitted by this material model (Figure 3a), an idealized constitutive relationship made of three linear branches was defined, as illustrated in Figure 3b. The first branch represents the initial elastic response, governed by the initial shear stiffness K_{in} , which controls the slope of the ascending branch and is required to determine the displacement corresponding to the maximum shear strength, denoted as $ePd3$. This quantity can be evaluated as the ratio between the maximum shear force V_{max} and the initial stiffness. The initial stiffness was computed by considering only the shear contribution, according to:

$$K_{in} = \frac{G \cdot A_i}{H} \quad (1)$$

where G is the shear modulus, A_i is the area of the considered structural member, and H is the corresponding column height or beam span length, depending on the element under examination. The same parameter was also used to define the residual stiffness, assumed equal to:

$$K_{res} = 0.8 \cdot K_{in} \quad (2)$$

The maximum shear strength was evaluated according to the shear resistance V_{res} model proposed by Sezen and Moehle in [72], which was calibrated on a broad experimental database [37,67,68] and explicitly accounts for the key parameters governing the shear capacity of lightly detailed reinforced concrete elements [37,73]. These include the dimensions of the member cross-section, the shear span, the concrete strength, the amount of transverse reinforcement, and the axial load acting on the element. According to this approach, the nominal shear strength is expressed as the sum of the concrete and transverse reinforcement contributions:

$$V_n = V_c + V_s \quad (3)$$

As far as the concrete contribution V_c is concerned, the adopted formulation is based on the assumption that diagonal tension governs the onset of shear cracking [74]. The initiation of the first inclined cracks is associated with the attainment of the principal tensile stress capacity under a plane stress condition in the $x - y$ plane. The principal tensile stress σ_1 can be written as:

$$\sigma_1 = \frac{\sigma_x + \sigma_y}{2} + \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau^2} \quad (4)$$

where σ_x and σ_y are the normal stresses along the x and y directions, respectively, while τ is the shear stress acting on the corresponding faces. Within this framework, the concrete shear contribution is linked to the tensile strength of concrete and to the stress state developing within the member prior to diagonal cracking, whereas the steel contribution V_s is associated with the action of transverse reinforcement crossing the inclined crack pattern. For reinforced concrete columns with poor transverse detailing, the confinement effect provided by stirrups can be considered negligible. Under this assumption, the stress component σ_x may be neglected, whereas the normal stress in the longitudinal direction σ_y of the member can be expressed as $-\frac{P}{A_l}$ where P is the axial load and A_l is the gross cross-sectional area of the column. By further assuming that diagonal cracking initiates when the principal tensile stress σ_1 reaches the tensile strength of concrete, taken as $f_{ct} = 0.5\sqrt{f'_c}$ (the concrete compressive strength in [MPa]), Equation (4) can be rearranged to explicitly derive the shear stress τ corresponding to crack initiation. This allows the concrete contribution to shear resistance to be related to the axial stress level and to the tensile capacity of the material, providing a rational estimate of the onset of diagonal tension failure:

$$\tau = 0.5\sqrt{f'_c} \cdot \sqrt{1 + \frac{P}{0.5\sqrt{f'_c} \cdot A_l}} \quad \text{in [MPa]} \quad (5)$$

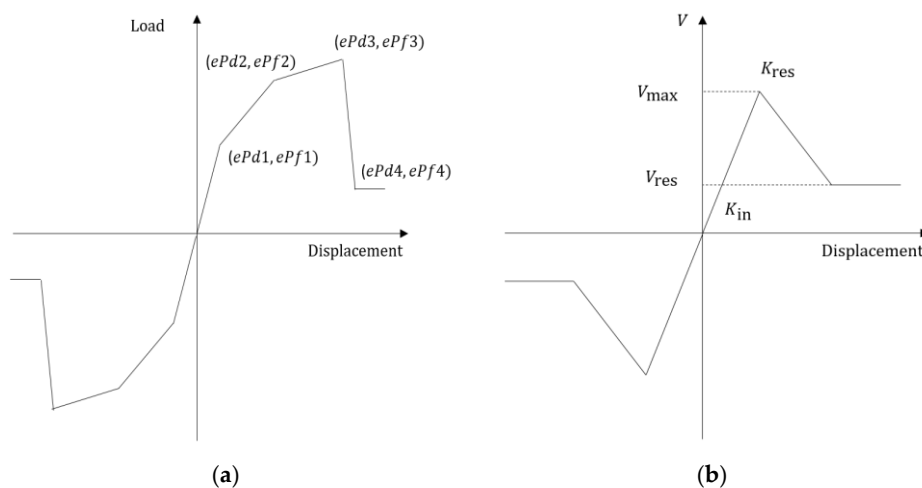


Figure 3. (a) Reference hysteretic response of the *Pinching4* material model implemented in OpenSees; (b) idealized trilinear constitutive law adopted for modeling the shear behavior of beam-column elements.

Let a denote the distance between the critical section, where the maximum bending moment develops, and the inflection point of the member, while d represents the distance between the extreme compression fiber of the section and the centroid of the tensile longitudinal reinforcement. According to the model proposed in the ASCE-ACI Committee 426 Report [75], the shear strength should be evaluated as a function of the inverse shear-span ratio $\frac{a}{d}$, with the recommended applicability range limited to $2 \ll \frac{a}{d} \ll 4$. Moreover, by assuming that only an effective area equal to $0.8 \cdot A_l$ contributes to the

shear-transfer mechanism, the concrete contribution to the nominal shear strength (moving from Equation (5)), V_c can be expressed as:

$$V_c = \frac{0.5\sqrt{f'_c}}{a/d} \cdot \sqrt{1 + \frac{P}{0.5\sqrt{f'_c} \cdot A_l}} \cdot 0.8A_g \quad \text{in [MPa]} \quad (6)$$

This formulation highlights the influence of both the axial compression level and the shear–span ratio on the concrete shear capacity, consistently with the experimental evidence on lightly reinforced existing reinforced concrete members.

With regard to the contribution of transverse reinforcement, the adopted approach follows the formulation provided in ACI 318-2002 [76], according to which the shear resistance offered by stirrups can be expressed as:

$$V_s = \alpha \frac{A_w f_y d}{s} \quad (7)$$

where A_w is the area of transverse reinforcement within spacing s , f_y is the yield strength of the stirrups, and d is the effective depth of the section. The coefficient α , which accounts for the inclination of shear cracks and the effectiveness of the transverse reinforcement, is taken equal to unity, consistently with both the ACI provisions [76] and the assumptions adopted in this study.

By combining the contributions of concrete (Equation (6)) and transverse reinforcement (Equation (7)), the nominal shear strength of the member (Equation (3)) can be expressed as:

$$\begin{aligned} V_n &= V_c + V_s = \\ &= \frac{A_w f_y d}{s} + \frac{0.5\sqrt{f'_c}}{a/d} \cdot \sqrt{1 + \frac{P}{0.5\sqrt{f'_c} \cdot A_l}} \cdot 0.8A_g \end{aligned} \quad (8)$$

This formulation enables a comprehensive evaluation of the shear capacity, accounting for both the material resistance and the structural detailing of the member. In the context of existing reinforced concrete buildings, where transverse reinforcement is often insufficient and poorly detailed, the contribution of V_s may be limited, making the accurate estimation of V_c and its degradation particularly critical for capturing the onset of brittle shear mechanisms. Furthermore, the maximum shear strength $V_{\max}$ is required to define the residual shear capacity, assumed equal to $V_{\text{res}} = 0.2 \cdot V_{\max}$. This parameter represents the post-peak shear resistance of the element and is used to capture the strength degradation associated with damage progression under cyclic loading.

The introduction of the shear links (Equation (8) and Figures 2b and 3) required substantial modifications to the original finite element model. In particular, the procedure culminated in the implementation of zero-length elements connecting duplicated nodes at the member ends so that the nonlinear spring could be placed between the original structural node and the node associated with the beam–column element. This strategy allowed the local shear deformability and degradation to be concentrated at the critical sections while preserving the global flexural behavior of the frame members. In this way, the numerical model became more suitable for capturing the expected response of existing gravity-load-designed reinforced concrete buildings, in which local brittle mechanisms may significantly influence the global seismic performance.

Overall, the adopted modeling approach provides a refined yet computationally manageable representation of the structure, capable of reproducing both distributed flexural nonlinearities and localized shear-sensitive response. This aspect is particularly important for the subsequent performance-based assessment, since the reliability of the estimated engineering demand parameters strongly depends on the ability of the numerical

model to represent the actual damage mechanisms governing the seismic behavior of the building.

The adopted numerical strategy is based on modeling approaches that have been extensively employed and validated in the scientific literature for the seismic assessment of reinforced concrete frame structures. In particular, the distributed-plasticity fiber formulation implemented in OpenSees has been widely recognized as an effective tool for simulating the nonlinear flexural behavior of reinforced concrete members, including stiffness degradation, strength deterioration, and plastic hinge spread. The reliability of this formulation has been demonstrated through numerous analytical and experimental benchmark studies available in the literature.

Particular attention was devoted to the representation of shear-sensitive mechanisms, which are known to play a critical role in gravity-load-designed reinforced concrete buildings constructed before the introduction of modern seismic provisions. For this reason, the conventional fiber model was complemented by nonlinear shear springs calibrated according to the shear strength model proposed by Sezen and Moehle, which was developed from an extensive experimental database of existing reinforced concrete columns. The introduction of these concentrated nonlinear elements allows the numerical model to reproduce the premature degradation and brittle failure mechanisms frequently observed in older reinforced concrete structures.

Furthermore, the consistency between the modal properties obtained from the numerical model, the expected dynamic characteristics of buildings belonging to the same structural typology, and the observed response during nonlinear static and dynamic analyses provides additional confidence in the capability of the model to realistically represent the global and local seismic behavior of the investigated structure. Consequently, the adopted modeling strategy was considered sufficiently robust for the purposes of performance-based seismic assessment and probabilistic loss estimation carried out in this study.

3. Structural Analyses

Following the definition and implementation of the numerical model described in the previous section, a comprehensive set of structural analyses was carried out to simulate the response of the building under both static and dynamic loading conditions. The adopted analysis framework is consistent with the requirements of PBEE [60,77,78], where structural response parameters are used as input for subsequent damage and loss assessment. The analytical procedure was organized in a progressive manner. First, linear modal analysis was performed to characterize the dynamic properties of the structure, including natural periods, mode shapes, and participating mass ratios. Subsequently, nonlinear static (pushover) analyses were conducted to investigate the global capacity and to identify the dominant failure mechanisms under increasing lateral loads. Finally, nonlinear time-history analyses were carried out by subjecting the structure to a set of ground motion records applied at the base, allowing for a more realistic representation of the seismic response under different intensity levels.

In particular, the dynamic analyses were performed using four sets of accelerograms characterized by increasing seismic intensity, selected and scaled according to an intensity-based approach consistent with the adopted PBEE framework [60,77,78]. This strategy enables the evaluation of structural response across a range of seismic demand levels, from moderate to severe events, thus providing the necessary input for probabilistic loss estimation.

The primary objective of this analysis phase is to determine the Engineering Demand Parameters (EDPs) required for the subsequent loss assessment carried out using the FEMA PACT [54,55]. Among these, the interstory drift ratio represents the key parameter,

as it is directly related to both structural and non-structural damage. Additional response quantities, such as floor accelerations, are also considered where relevant.

The results obtained from the different analyses are therefore used to establish a consistent link between seismic intensity measures and structural response, forming the basis for the derivation of fragility functions and the estimation of economic losses. The following subsections describe in detail the modal analysis (Section 3.1), the nonlinear static analysis (Section 3.2), and the nonlinear dynamic time-history analyses (Section 3.3).

3.1. Modal Analysis

Modal analysis was performed as a preliminary step to characterize the dynamic behavior of the structure and to identify its fundamental vibration properties. For a complex reinforced concrete building such as the one investigated in this study, modal analysis plays a key role in predicting the natural periods of vibration and the associated mode shapes. These parameters are essential for understanding the global dynamic response of the structure when subjected to seismic excitation. The analysis was carried out using the developed OpenSees numerical model [65], assuming linear elastic behavior. The resulting natural periods and modal properties are summarized in Table 1. These results provide a first insight into the dynamic characteristics of the building and serve as a reference for both the nonlinear static and dynamic analyses described in the following sections.

Table 1. Natural periods and corresponding frequencies of the first three vibration modes obtained from the modal analysis of the building.

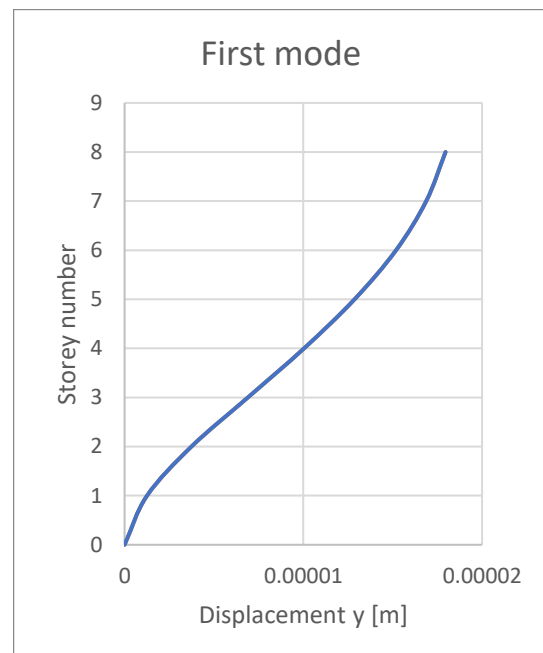
Mode	Period [Sec]	Frequency [Hz]
1	2.283	0.438
2	1.337	0.748
3	1.273	0.785

To better interpret the structural response, the eigenvectors corresponding to the dominant modes were extracted with reference to a master node, identified at the level of a central column of the structure. This approach allows for a clear representation of the global deformation patterns and facilitates the identification of the governing modes in each principal direction. The results indicate that the predominant flexural behavior in the two main horizontal directions is associated with the first and third modes of vibration. In particular, the first mode (Figure 4) is characterized by a translational response primarily along one principal direction (e.g., y -direction), while the third mode (Figure 5) governs the flexural response in the orthogonal direction (x -direction). Conversely, the second mode is mainly torsional, involving rotation of the structure around its vertical axis with limited contribution to the global translational response.

These findings highlight a non-negligible coupling between translational and torsional behavior, which is typical of existing buildings with irregular stiffness and mass distribution. The identification of the dominant modes is particularly relevant for the subsequent analyses, as it supports the selection of appropriate lateral load patterns for push-over analysis and contributes to the interpretation of dynamic response under seismic excitation.

Storey number	Displacement y [m]
8	0.000017957
7	0.000016875
6	0.000015188
5	0.000012866
4	0.000010039
3	6.8969E-06
2	3.7412E-06
1	1.2563E-06
0	0

(a)

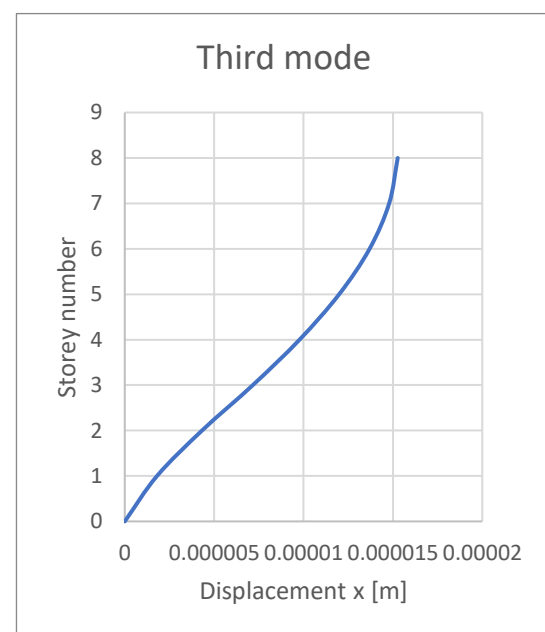


(b)

Figure 4. (a) Storey-wise displacement profile in the y -direction associated with the first vibration mode; (b) graphical representation of the first mode shape, showing the predominant flexural behavior along the y -direction.

Storey number	Displacement y [m]
8	0.000017957
7	0.000016875
6	0.000015188
5	0.000012866
4	0.000010039
3	6.8969E-06
2	3.7412E-06
1	1.2563E-06
0	0

(a)



(b)

Figure 5. (a) Storey-wise displacement profile in the x -direction associated with the third vibration mode; (b) graphical representation of the third mode shape, showing the predominant flexural behavior along the x -direction.

3.2. Pushover Analysis

The nonlinear static (pushover) analysis was performed to evaluate the global capacity of the structure and to identify the main failure mechanisms under increasing lateral loads. This procedure consists of an incremental and iterative solution of the nonlinear equilibrium equations, typically based on the Newton–Raphson algorithm, applied to a

structural model subjected to monotonically increasing horizontal forces. Although simplified, pushover analysis provides a useful approximation of the structural response under seismic actions, with significantly reduced computational effort compared to nonlinear time-history analysis. Moreover, it does not require the definition of input ground motion records and allows for a straightforward interpretation of results, since displacements and inelastic deformations can be directly related to damage.

The applied load patterns consist of lateral force distributions that are progressively scaled during the analysis through an automatically updated load factor until structural collapse or a predefined displacement limit is reached. According to the Italian seismic code (NTC 2018) [79] and European code [80], at least two categories of load distributions should be considered: a primary distribution (Group 1) and a secondary distribution (Group 2). The Group 1 distribution can be defined either as proportional to the equivalent static forces or to the fundamental mode shape, provided that the participating mass ratio of the fundamental mode exceeds 75%. Group 2 distributions may include uniform, adaptive, or multimodal load patterns, the latter typically involving a combination of several significant vibration modes. In this study, two pushover analyses were carried out by adopting only the Group 1 load pattern, defined as proportional to the modal shape governing each principal direction. Specifically, in the x -direction (Figure 6a), the lateral load distribution was derived from the third mode of vibration, while in the y -direction (Figure 6b) it was based on the first mode, consistently with the modal analysis results presented in Section 3.1. This choice reflects the dominant flexural behavior observed in the corresponding directions and ensures a more realistic representation of the expected deformation pattern.

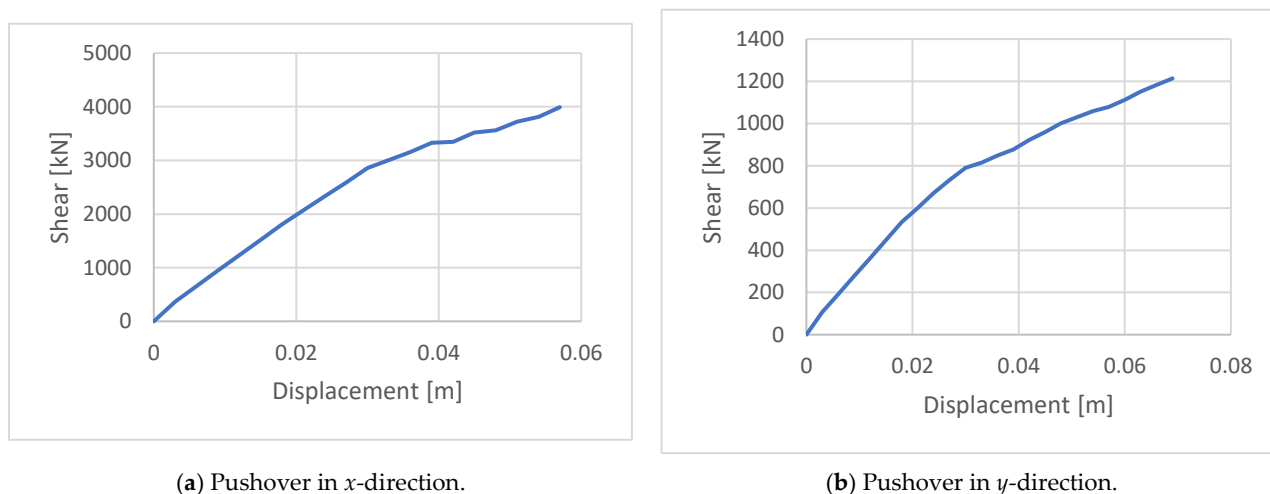


Figure 6. (a) Pushover capacity curve in the x -direction obtained using a lateral load distribution proportional to the third vibration mode; (b) pushover capacity curve in the y -direction obtained using a lateral load distribution proportional to the first vibration mode.

The analyses were conducted to estimate the horizontal capacity of the structure and to investigate its nonlinear response prior to collapse. The resulting capacity curves (Figure 6) indicate that the structural response is strongly influenced by shear-sensitive behavior. In particular, the structure is unable to fully develop its theoretical shear capacity due to the progressive failure of the shear links introduced in the numerical model (Section 2). The accumulation of local shear failures leads to numerical convergence issues, ultimately interrupting the analysis before a global collapse mechanism can be reached. This behavior highlights the critical role of shear mechanisms in existing reinforced concrete buildings designed for gravity loads, where insufficient transverse reinforcement and poor detailing

may lead to premature brittle failures. The same trend was also observed in several nonlinear dynamic analyses, confirming that local shear degradation significantly affects the global seismic performance of the structure. These findings underline the importance of incorporating refined shear modeling in the numerical analysis, especially when the results are intended for performance-based loss assessment.

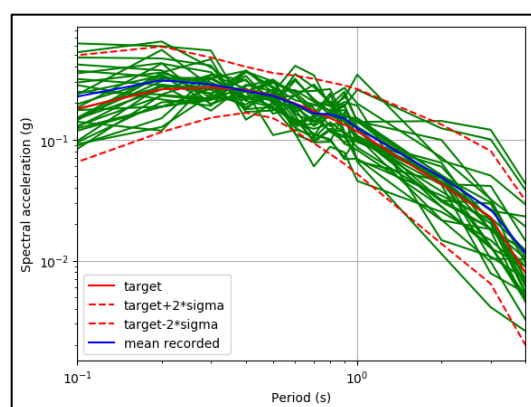
It should be emphasized that the influence of shear degradation is not necessarily reflected by a sudden reduction in the global lateral capacity. Owing to the redundancy of the structural system, local shear failures may be partially compensated by force redistribution among adjacent members. Nevertheless, the progressive deterioration of the shear links significantly affects stiffness degradation, displacement demand, and numerical convergence, particularly under increasing seismic intensity. Similar observations have been reported in [81], who highlighted the importance of accounting for shear-sensitive mechanisms in the estimation of both structural capacity and seismic demand.

3.3. Time History Analysis

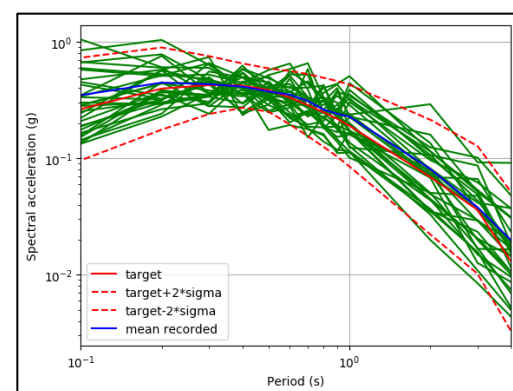
Among the available analysis procedures, nonlinear dynamic time-history analysis represents the most comprehensive approach, as it allows for a direct simulation of the structural response under realistic seismic input. However, it is also the most computationally demanding and complex method, requiring the selection and scaling of appropriate ground motion records as well as the definition of a refined nonlinear structural model. In the present study, nonlinear time-history analyses were performed by applying to the structure four sets of accelerograms, defined consistently with the InfraNAT project [82] framework, and characterized by different levels of seismic intensity associated with increasing probabilities of exceedance, as summarized in Table 2. Each set consists of 60 individual records, including 30 accelerograms applied along the principal X-direction of the building and 30 along the Y-direction. The corresponding response spectra for each intensity level are shown in Figure 7, highlighting the variation in seismic demand with increasing return periods.

Table 2. Definition of the four ground motion sets adopted in the nonlinear time-history analyses, expressed in terms of probability of exceedance in 50 years and corresponding seismic intensity levels.

-	Return Period [Years]	Probability of Exceedance
1	98	40% in 50 years
2	224	20% in 50 years
3	475	10% in 50 years
4	975	5% in 50 years



(a) 98 years.



(b) 224 years.

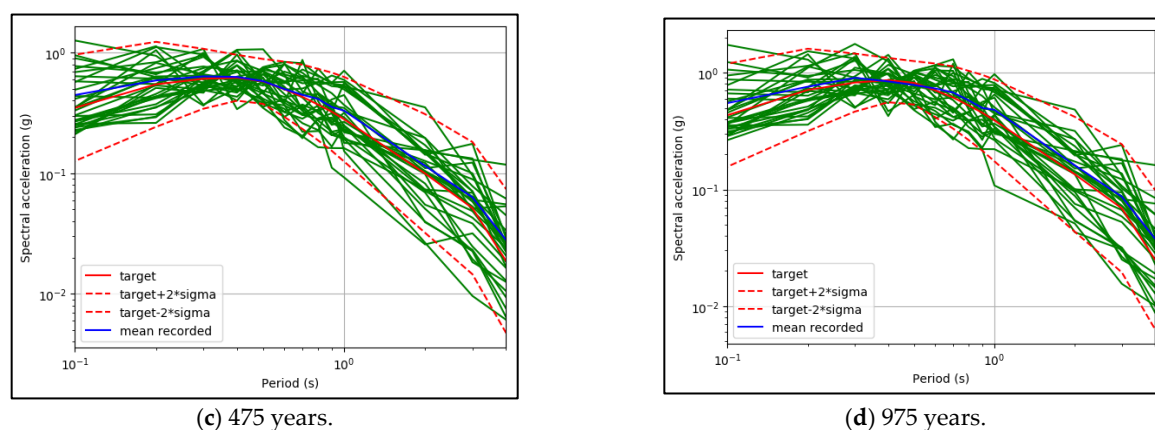


Figure 7. Mean and dispersion of the acceleration response spectra of the selected ground motion sets for the four seismic intensity levels considered, defined according to different probabilities of exceedance: (a) 98 years; (b) 224 years; (c) 475 years; (d) 975 years.

A total of 120 pairs of bidirectional ground motions were considered, corresponding to 30 pairs for each of the four intensity levels. These analyses provided a statistically robust dataset of structural responses, enabling the evaluation of demand parameters across a wide range of seismic scenarios. For each intensity level, the maximum interstory drift ratio (IDR) profiles were computed, representing the primary Engineering Demand Parameter (EDP) adopted in this study. The IDR is widely recognized as one of the most effective indicators of structural and non-structural damage [27,45,83], as it directly correlates with damage in beam–column joints, columns, beams, and infill panels. The focus on interstory drift is consistent with the objectives of the present research, which aims to estimate the economic losses associated with earthquake-induced damage. Indeed, both structural and non-structural components contribute significantly to the total repair cost, and their damage states can be effectively related to drift-based fragility functions within the PBEE framework. In addition to IDR, floor acceleration responses were also evaluated, as they are particularly relevant for the assessment of acceleration-sensitive non-structural components and equipment [36,43]. The reliability of nonlinear dynamic analyses strongly depends on the representativeness of the selected ground-motion records and their consistency with the seismic hazard characteristics of the site. Recent studies have highlighted the importance of adopting record-selection procedures capable of ensuring both spectral compatibility and an adequate representation of record-to-record variability, particularly in the context of multi-directional seismic analyses [84]. In the present study, the adopted record sets were selected within the broader InfraNAT framework and grouped according to four hazard-consistent intensity levels, allowing the variability of structural response and loss estimates to be captured through a large number of nonlinear dynamic simulations.

The main results of the nonlinear dynamic analyses are summarized in Figure 8. In particular, Figure 8a,b present the maximum interstory drift ratios in the X- and Y-directions, respectively, while Figure 8c,d show the corresponding floor accelerations in the two principal directions. These results provide a comprehensive description of the structural demand and constitute the fundamental input for the derivation of fragility functions and the evaluation of economic losses through the FEMA PACT methodology [54,55].

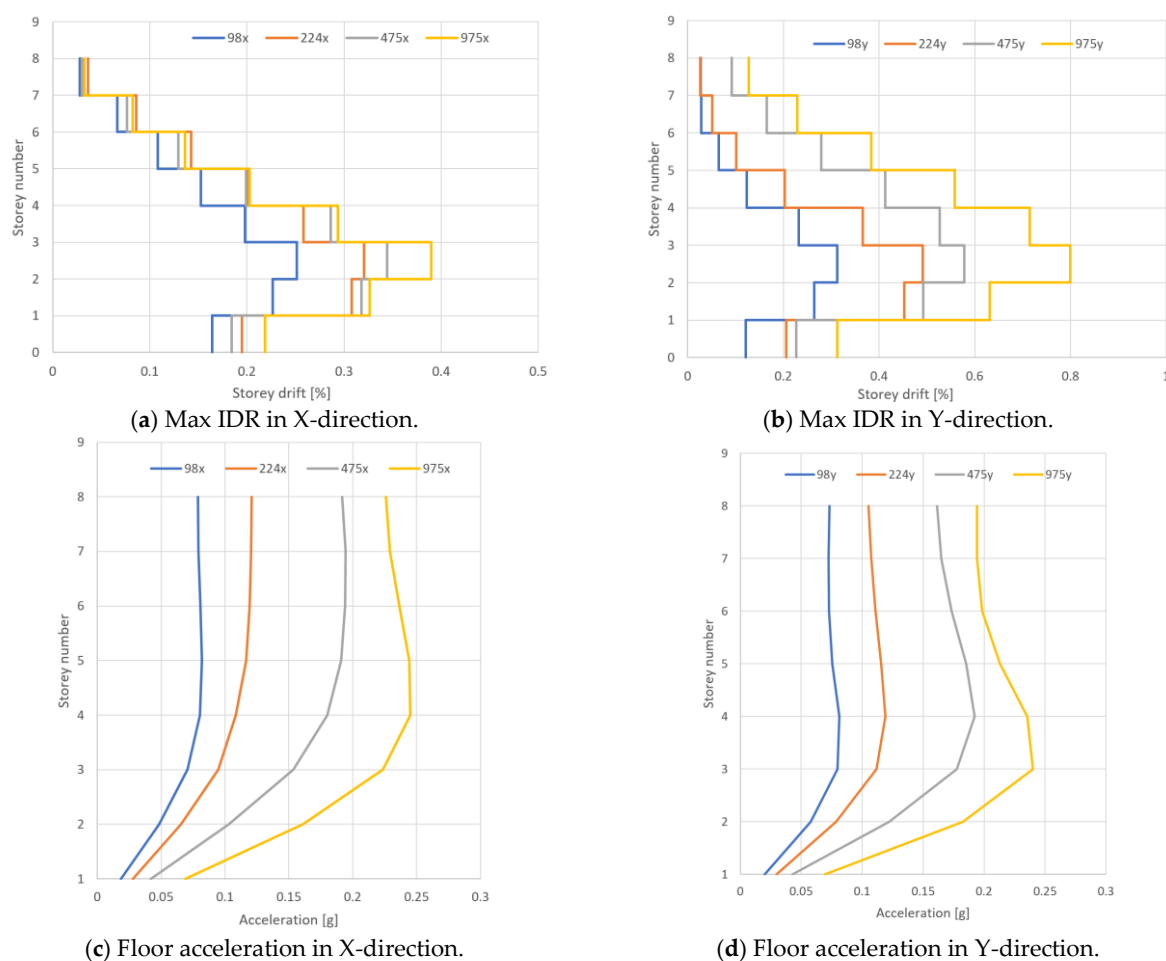


Figure 8. (a,b) Maximum interstory drift ratio (IDR) envelopes along the building height in the X- and Y-directions, respectively; (c,d) floor acceleration envelopes in the X- and Y-directions, derived from nonlinear time-history analyses.

4. PBEE/PACT Modeling and Loss Assessment

The estimation of seismic-induced economic losses was carried out within the framework of PBEE [77,78] through the implementation of the Performance Assessment Calculation Tool (PACT) developed in the context of the FEMA P-58 methodology [54,55]. PACT is a computational platform that performs probabilistic loss calculations by integrating seismic hazard, structural response, damage models, and consequence functions. Within this framework, structural analysis results are translated into quantitative performance metrics, expressed in terms of economic losses, repair times, and potential casualties. The first step in the PACT implementation consists of defining the main characteristics of the building, as described in Section 2. These include geometric properties, structural typology, number of stories, and general construction features. Such information is necessary to correctly represent the building within the PBEE framework and to associate it with appropriate fragility functions for both structural and non-structural components.

Unlike conventional seismic assessment procedures, which are primarily focused on verifying structural safety through force- or displacement-based acceptance criteria, the FEMA P-58 methodology allows the consequences of earthquake damage to be quantified in probabilistic and economic terms. The framework is based on the Performance-Based Earthquake Engineering (PBEE) philosophy developed by PEER, in which seismic hazard, structural response, damage occurrence, and economic consequences are linked through a sequence of probabilistic models. Within this framework, the nonlinear analyses described in Section 3 provide the Engineering Demand Parameters (EDPs), primarily represented by

interstory drift ratios. These quantities are subsequently combined with component fragility functions to estimate the probability of reaching different damage states. The damage states are then associated with consequence functions describing repair actions and repair costs. Finally, the resulting losses are integrated with the site-specific seismic hazard curve to obtain time-based risk metrics, including the Expected Annual Loss (EAL). The adoption of FEMA P-58 is particularly advantageous for existing buildings because it enables structural performance to be expressed in terms that are directly understandable by owners, stakeholders, insurance companies, and public administrations. Rather than simply stating whether a building satisfies a prescribed safety requirement, the methodology quantifies the expected economic consequences associated with earthquake damage and therefore provides a rational basis for risk-informed decision-making and retrofit prioritization.

A fundamental input parameter is the core and shell replacement cost, which represents the best estimate of the cost required to replace the building's primary structural and architectural components. This value includes the cost of demolition, debris removal, and reconstruction of all core and shell elements, such as structural frames, floors, façades, staircases, elevators, and essential mechanical and electrical systems. Given the hospital function of the building, the cost estimation was performed by distinguishing between two main functional categories: healthcare-related areas (including wards, operating rooms, intensive care units, and medical services) and non-healthcare areas (such as circulation spaces, technical rooms, and general services). Based on values reported in the scientific literature [85,86], unit construction costs were assumed equal to approximately 2000 €/m² for healthcare areas and 1800 €/m² for non-healthcare areas. Considering a typical floor area of approximately 494 m², for the building under consideration (Section 2) the total construction cost of the building (core and shell) was estimated at about 8 million euros. This value does not include centralized technological systems serving the hospital complex (e.g., heating and cooling plants, medical gas systems, electrical substations, water treatment facilities), nor additional costs such as taxes, technical fees, administrative expenses, and contingencies. Since hospitals operate as interconnected systems and rely on shared infrastructure, it was assumed that these technological systems contribute an additional 20% to the building cost. This leads to an estimated value of approximately 2 million euros, resulting in a total replacement cost rounded to 10 million euros for the purposes of the analysis.

Another essential component of the PBEE framework is the definition of the population model, which is required to estimate potential casualties and to properly account for occupancy-dependent losses. In this study, the distribution of occupants was defined as a function of the time of day. A baseline occupancy of approximately 2 persons per 93 m² was assumed over the entire day, representing patients and minimal staff presence. During working hours, an additional occupancy of approximately 3 persons per 93 m² was considered, corresponding to medical and support personnel, resulting in a peak occupancy of 5 persons per 93 m². The adopted occupancy profile is illustrated in Figure 9.

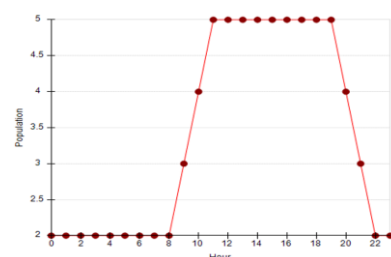


Figure 9. Occupancy model adopted for the loss assessment, showing the variation in occupant density within the building as a function of time, with baseline and peak conditions representative of hospital usage.

Following the definition of the general building characteristics, a key step in the PBEE framework consists of quantifying the type, quantity, vulnerability, and spatial distribution of damageable components within the structure. This is achieved through the adoption of component-based fragility and consequence models, which relate structural response parameters to probabilistic damage and loss outcomes. In the present study, fragility and loss data were derived from literature sources representative of reinforced concrete buildings constructed prior to the 1970s, i.e., before the introduction of modern seismic design provisions [56]. These buildings are typically characterized by gravity-load design, low-strength concrete, and inadequate detailing of reinforcement, making them particularly vulnerable to seismic actions.

Three main Fragility Groups (FGs) were considered to represent the structural components governing the seismic response of pre-1970 RC frame buildings:

- External beam–column joints (EWJs);
- Internal beam–column joints (IWJs);
- Weak ductile columns (DWCs).

For each fragility group, a set of discrete Damage States (DSs) was defined to describe the progression of damage under increasing seismic demand. The damage states are associated with both local material degradation and global structural response indicators. In particular, three primary damage states were considered:

- **DS1 (slight damage):** characterized by the onset of localized cracking in concrete and initial yielding of longitudinal reinforcement, typically associated with residual crack widths smaller than approximately 1–1.5 mm;
- **DS2 (moderate damage):** associated with the propagation and widening of cracks, including initial concrete fragmentation, with crack widths generally ranging between 3 mm and 5 mm;
- **DS3 (severe damage):** corresponding to significant concrete spalling, exposure of longitudinal and transverse reinforcement, and degradation of structural capacity, typically involving 20–30% of the component surface.

A further damage condition (DS4) can be associated with incipient or near-collapse behavior, representing the ultimate limit state of the component.

The fragility functions were calibrated based on experimental datasets selected from [56], referring to test specimens representative of typical 1970s RC construction. Within the PBEE framework, the EDP adopted to describe damage evolution is the IDR consistently with the results obtained from nonlinear dynamic analyses (Section 3.3). The IDR is particularly suitable for capturing both structural and non-structural damage in frame systems. The experimental database compiled in [56] includes cyclic tests performed on reinforced concrete beam–column joints and columns representative of existing gravity-load-designed buildings constructed before the introduction of modern seismic design provisions. The selected specimens are characterized by low-strength concrete, inadequate transverse reinforcement, and detailing practices commonly adopted in reinforced concrete buildings built before the 1970s. Based on the observed damage progression during testing, the authors identified statistically significant relationships between interstory drift ratio (IDR) and the probability of exceeding predefined damage states. Lognormal fragility functions were subsequently calibrated for each fragility group by evaluating the median drift capacity and the associated dispersion corresponding to the occurrence of each damage state. Since the investigated hospital building belongs to the same construction period and structural typology, these fragility relationships were considered representative and were therefore adopted in the present study.

The parameters defining the fragility functions for each component group are reported in Table 3, while the corresponding fragility curves are shown in Figure 10. These curves express the probability of exceeding a given damage state as a function of the imposed IDR.

Table 3. Statistical parameters of the fragility functions (median drift capacity and logarithmic standard deviation) for each damage state (DS1–DS3) and component group (EWJ, IWJ, DWC).

Fragility Group	Damage State	Fragility Function Parameters			
		Median IDR (%)	β_u	Dispersion β_r	β
EWJ	DS1	0.55	0.23	0.35	0.4
	DS2	1.5	0.25	0.3	0.4
	DS3	2.75	0.26	0.3	0.4
IWJ	DS1	0.65	0.24	0.3	0.4
	DS2	1.75	0.14	0.3	0.35
	DS3	3	0.07	0.3	0.3
DWC	DS1	0.75	0.07	0.35	0.4
	DS2	1.75	0.1	0.3	0.35
	DS3	3	0.12	0.3	0.35

In Table 3 the parameter β_r represents the record-to-record variability observed in the experimental data from which the fragility parameters are derived. The parameter β_u accounts for the uncertainty associated with the extent to which the experimental tests accurately represent real installation and loading conditions of components within an actual building, as well as the uncertainty related to the limited size of the available dataset in capturing the full variability of the response. The total dispersion, β , is then evaluated as the square root of the sum of squares (SRSS) of the two aforementioned parameters.

Following the definition of fragility functions, the next step within the PBEE framework is the characterization of loss functions. The economic losses associated with the repair of damaged reinforced concrete components can be expressed through probabilistic loss models, which relate each damage state to a corresponding repair cost. Loss functions are defined as probabilistic estimates of the costs associated with repair or replacement actions required for individual structural components once a specific damage state is reached. In other words, they provide the probability distribution of economic loss conditioned on the occurrence of a given damage state. This approach allows for a direct linkage between structural damage and its economic consequences. In the present study, each damage state was uniquely associated with a predefined set of repair interventions required to restore the structural component to its pre-earthquake (essentially undamaged) condition. These interventions include, depending on the severity of damage, operations such as crack injection, localized strengthening, partial replacement of damaged elements, or full component substitution.

In a subsequent step, unit repair costs were estimated for each intervention (see Table 4), based on available data from the literature and on cost analyses of representative pre-1970 reinforced concrete frame buildings [56,87]. From these values, normalized repair cost ratios were derived, enabling a consistent comparison across different component types and damage states.

It should be noted that the adopted repair costs are not directly derived from the explicit quantification of individual damage manifestations obtained from the numerical model. Rather, the nonlinear fiber-based analyses provide the Engineering Demand Parameters (EDPs), expressed in terms of interstory drift ratios, which are subsequently used to evaluate the probability of occurrence of predefined damage states through the adopted fragility functions. Each damage state is then associated with a specific repair strategy and corresponding repair cost. Consequently, the total expected repair cost is obtained

through the combination of structural response, fragility assessment, and component loss functions within the PBEE framework.

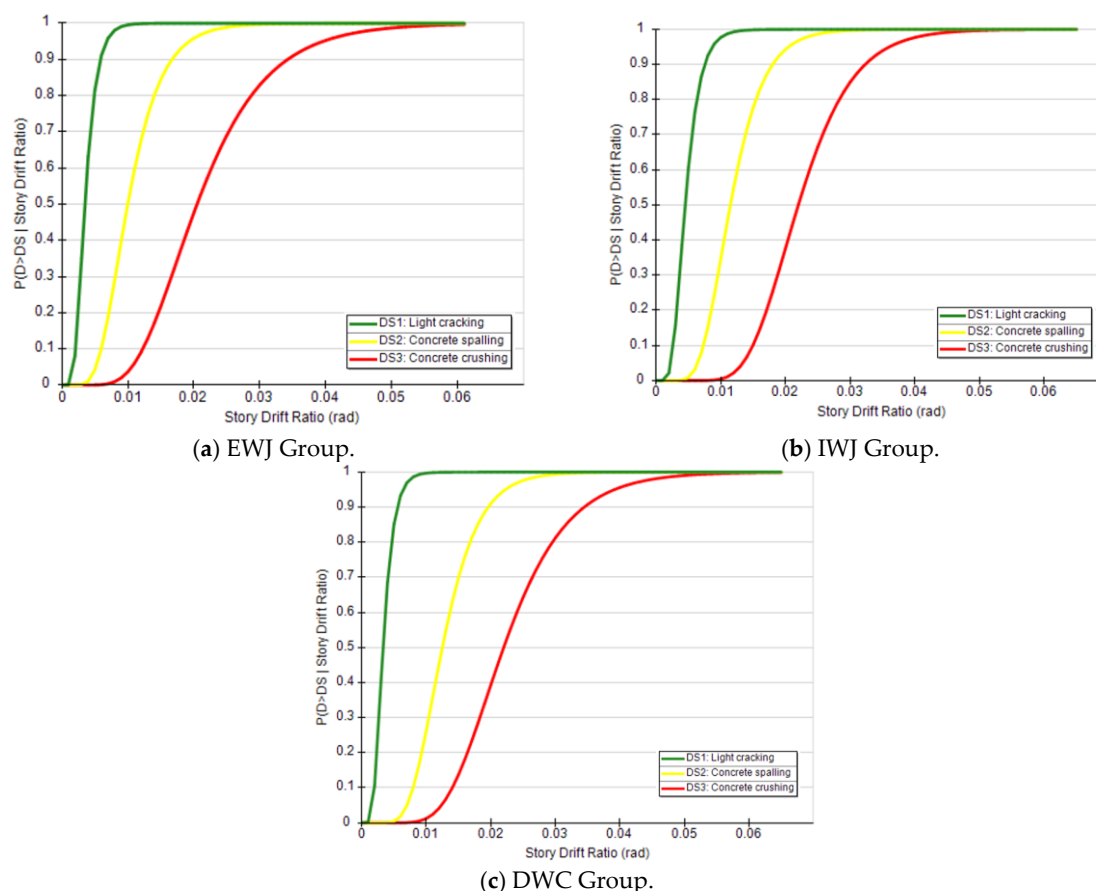


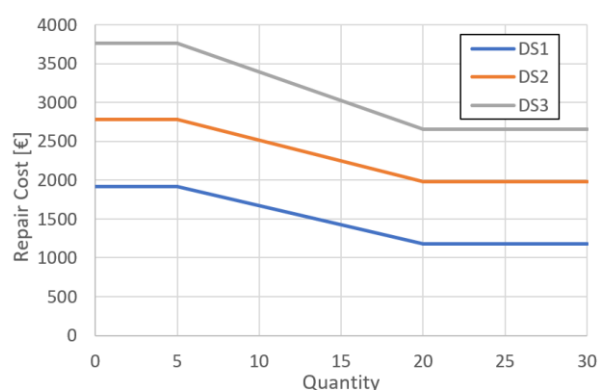
Figure 10. Fragility curves expressing the probability of exceeding different damage states (DS1–DS3) as a function of interstory drift ratio (IDR) for: (a) external beam–column joints (EWJs); (b) internal beam–column joints (IWJs); (c) weak ductile columns (DWCs).

The repair-cost distributions shown in Figure 11 were obtained by combining the unit repair costs listed in Table 4 with the corresponding quantities of repair work associated with each component group and damage state. In this context, the term “quantity” refers to the amount of repair intervention required for each action, expressed according to the unit reported in Table 4, such as treated surface area, injected crack length, or number of replaced reinforcement bars. For each damage state, the cost of each repair action was calculated as the product of the unit cost and the corresponding quantity. The sum of these contributions provided the total repair cost for the considered component group. The values reported in Figure 11 were then normalized with respect to the total repair cost of the corresponding damage state, so that the figure represents relative cost contributions rather than absolute repair costs.

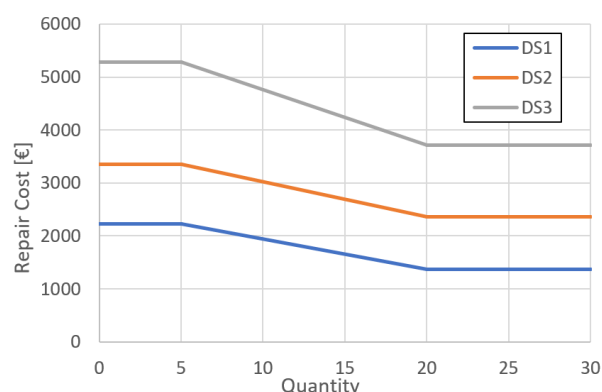
Table 4. Repair actions and corresponding unit costs adopted for each damage state (DS1–DS3), used to derive normalized loss functions for reinforced concrete components.

Damage State	Repair Action	Unit	Unit Cost [€/Unit]
DS1 (Light cracking)	Cleaning of the area adjacent to cracks	m ²	6.61
	Preparation for crack injection	m ²	27.01
	Epoxy resin injection	m	156.00
DS2	Removal of deteriorated concrete and crack cleaning	m ²	6.61

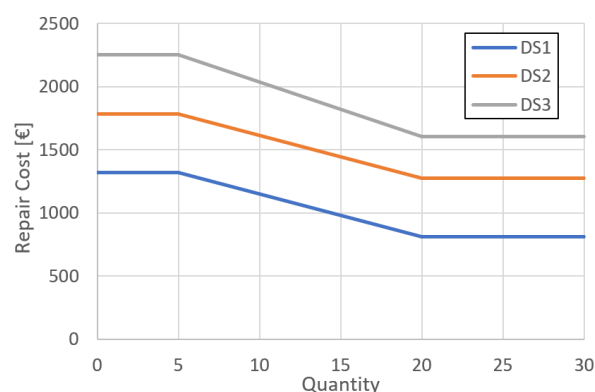
DS3 (severe cracking) (spalling, concrete crushing, re-bars buckling)	Preparation of cracks/surfaces for injection/repair	m ²	27.01
	Epoxy resin injection	m	156.00
	Repair of deteriorated concrete using mortar	m ²	88.35
	Removal of deteriorated concrete and crack cleaning	m ²	6.61
	Surface preparation to ensure proper bonding between new and existing concrete	m ²	27.01
	Epoxy resin injection	m	156.00
	Replacement of deformed reinforcement bars	cad	50.00
	Replacement of damaged concrete using rheoplastic concrete or high-performance mortar	m ²	109.26



(a) EWJ Group.



(b) IWJ Group.



(c) DWC Group.

Figure 11. Normalized repair-cost contributions for the considered structural component groups—external beam–column joints (EWJs), internal beam–column joints (IWJs), and weak ductile columns (DWCs)—obtained by combining unit repair costs and repair quantities for each damage state (DS1–DS3).

The distribution of repair costs among the considered fragility groups, external beam–column joints (EWJs), internal joints (IWJs), and weak ductile columns (DWCs), is illustrated in Figure 11 for each damage state. As expected, repair costs increase with the severity and extent of damage, moving from DS1 to DS3. However, the relative contribution of individual components remains bounded, with repair costs not exceeding approximately 16% of the total at DS1 and about 30% at DS3. The most significant cost contributions are associated with repair/replacement activities and safety-related operations, which together account for approximately 50% of the total repair cost, regardless of the damage state considered. These findings highlight the importance of both direct intervention costs and ancillary operations (e.g., temporary stabilization, safety measures) in the overall economic impact of seismic damage.

It is important to distinguish between the component fragility functions used for repairable damage assessment and the collapse fragility function used for global loss estimation. The fragility functions reported in Table 3 and Figure 10 are EDP-based relationships, expressed as a function of interstory drift ratio. They are used to estimate the probability of exceeding different damage states for each structural component group and therefore control the repair costs associated with non-collapse realizations. Conversely, the collapse fragility function shown in Figure 12a is an IM-based relationship, expressed as a function of the selected seismic intensity measure. It describes the probability of global collapse of the building and is used to account for collapse-related consequences, typically associated with replacement-level losses. Therefore, Figure 12a is not directly derived from the component fragility functions in Figure 10, but it complements them within the FEMA P-58/PACT framework.

To complete the PBEE framework, two additional components are required: the collapse fragility function and the seismic hazard function. A collapse fragility function is introduced to quantify the probability of global structural failure, which is directly associated with severe damage scenarios, including potential injuries and fatalities within the collapse area. The collapse fragility function (Figure 12a) expresses the probability of building collapse as a function of seismic intensity, taking into account one or more relevant vibration modes (Section 3.1) and incorporating the results of both nonlinear static (Section 3.2) and nonlinear dynamic analyses (Section 3.3). This function represents a fundamental element of the PBEE framework, as it links the structural response to extreme damage states and enables the estimation of collapse-related consequences. The final step prior to the evaluation of results is the definition of the seismic hazard curve. This curve describes the probability of exceeding a given level of ground motion intensity at a specific site within a defined time interval. In other words, seismic hazard analysis provides the relationship between seismic intensity measures and their corresponding probabilities of exceedance over a reference period. In the present study, the seismic hazard was expressed in terms of an intensity measure based on the average spectral acceleration, consistently with the approach adopted in the nonlinear dynamic analyses (Section 3.3). This allowed for the construction of the hazard curve shown in Figure 12b, which establishes the probabilistic occurrence of different intensity levels and serves as a key input for the computation of expected losses.

The combination of fragility functions, loss functions, and seismic hazard representation enables the full implementation of the PBEE methodology, ultimately leading to the estimation of the EAL presented in the following section.

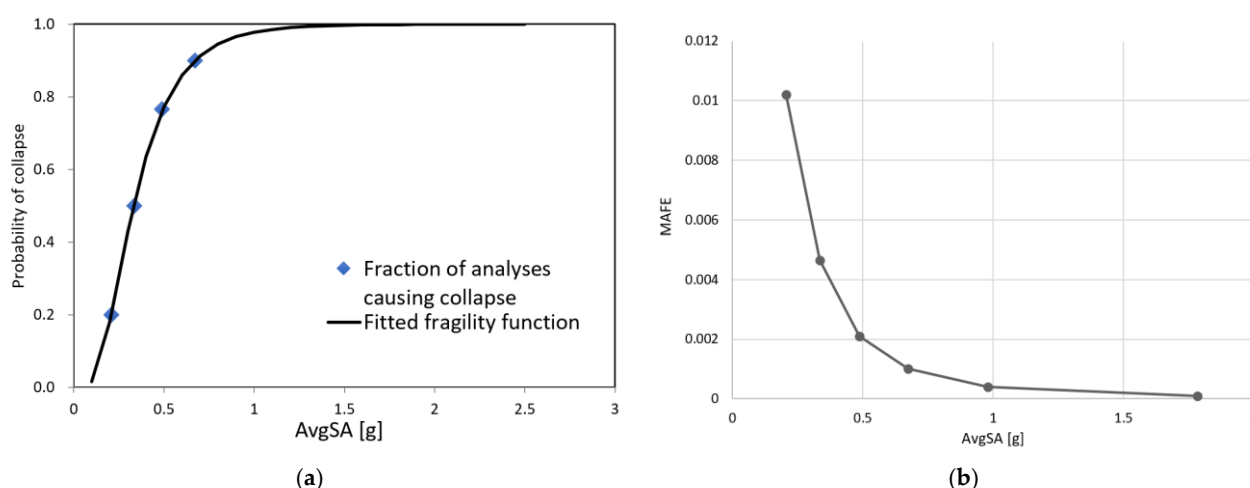
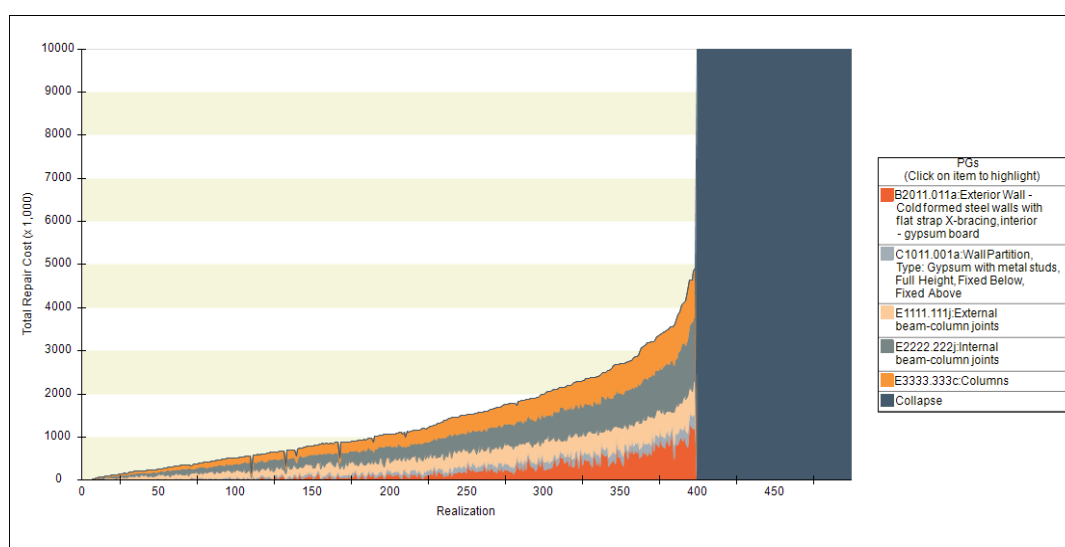


Figure 12. (a) Collapse fragility function expressing the probability of global structural collapse as a function of seismic intensity; (b) seismic hazard curve at the site in terms of mean annual frequency of exceedance (MAFE) of the selected intensity measure (AvgSA).

5. Results and Expected Annual Loss Assessment

The results presented in this section are structured to support the needs of decision-makers, who may require the comparison of different design or retrofit strategies in terms of seismic performance. This is particularly relevant both for the design of new buildings and for cost–benefit analyses aimed at evaluating the effectiveness of structural strengthening or retrofiting interventions. Such forward-looking economic scenarios are of increasing interest for financial institutions and insurance companies, as they enable a quantitative assessment of seismic risk. The analysis is first conducted in an intensity-based framework, where the expected losses associated with both structural and non-structural components are computed for each considered seismic intensity level. These results are subsequently combined with the seismic hazard curve of the site, allowing the transition to a time-based representation of losses. In this way, economic losses can be expressed as a function of the return period associated with each seismic event. The ultimate objective is to translate the seismic risk of the building into the EAL, which represents the average annual economic loss associated with the seismic exposure and vulnerability of the structure. Based on the information obtained from nonlinear dynamic analyses (in terms of IDR), together with the adopted fragility functions, loss functions, seismic hazard curve, and collapse fragility function, it is possible to evaluate the probabilistic distribution of losses for each intensity level.

Figure 13 illustrates the distribution of repair costs obtained for each seismic intensity level. It can be observed that, as the intensity of the seismic input increases, the number of simulations leading to collapse progressively increases, consistently with the results obtained from nonlinear time-history analyses. For the realizations not resulting in collapse, it is possible to quantify the contribution of individual structural components, namely, external beam–column joints (EWJs), internal joints (IWJs), and weak ductile columns (DWCs), as well as non-structural components, such as external and internal partition systems, to the total repair cost. Considering, for example, Figure 13a, which reports 500 realizations ordered in ascending cost, the repair cost corresponding to a 50% probability of non-exceedance can be identified as the value associated with the median realization (i.e., the 250th). In this case, the total repair cost (TRC—Total Repair Cost) is approximately equal to 1,514,285.71 €. The same information can be more clearly interpreted in Figure 14, where the discrete realizations are transformed into a continuous curve representing the probability that the total repair cost does not exceed a given economic loss level (EL—Economic Loss).



(a)

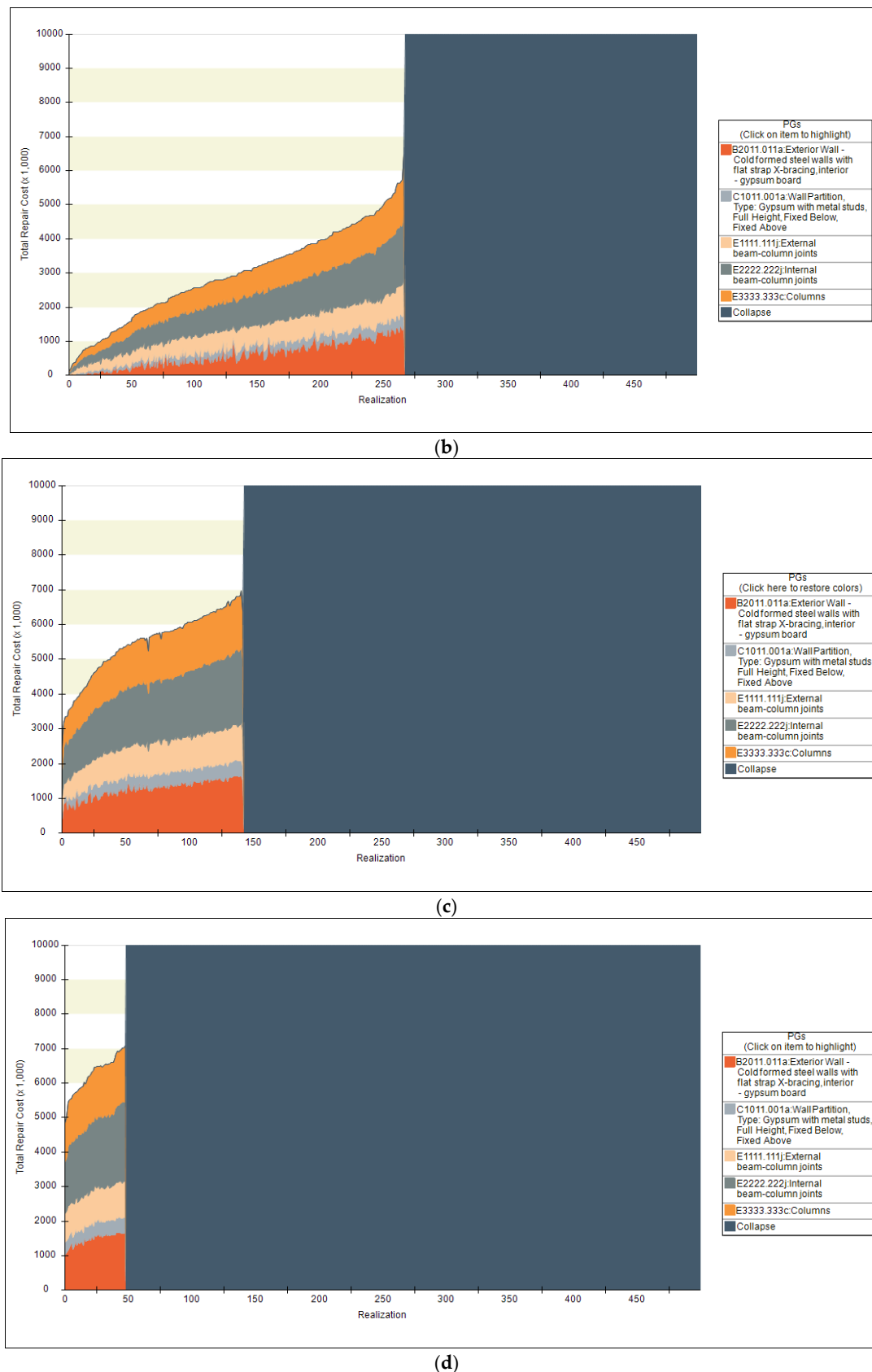


Figure 13. Distribution of total repair costs (TRC) obtained from Monte Carlo simulations for each seismic intensity level, including both collapse and non-collapse realizations; the progressive increase in collapse occurrences with increasing intensity is highlighted. Total repair cost for return periods of 98 (a), 224 (b), 475 (c), and 975 years (d) derived from 500 realizations sorted in ascending order (Monte Carlo simulations). The realizations are statistically independent Monte Carlo samples and should therefore be interpreted as possible outcomes of the same seismic intensity level.

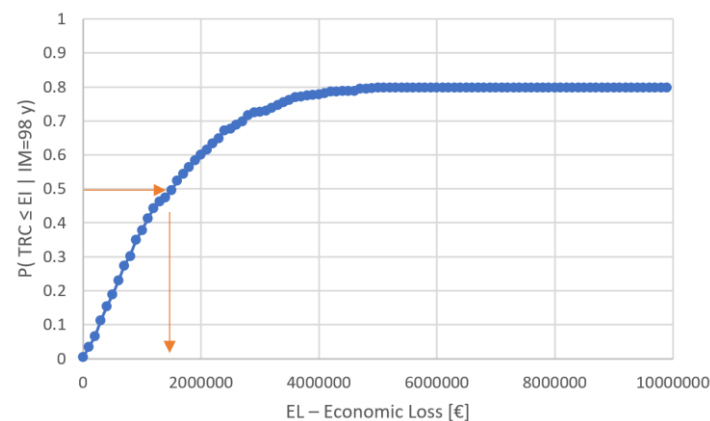


Figure 14. Probabilistic loss curve expressing the probability of non-exceedance of the total repair cost (TRC) as a function of economic loss (EL), derived from the ensemble of simulated realizations.

To better highlight the actual economic losses obtained, Figure 15a presents the histograms of repair costs corresponding to a 50% probability of non-exceedance for each selected intensity level along the horizontal axis. It can be observed that, for seismic events associated with return periods greater than 224 years, the expected loss approaches the total replacement cost, as the probability of structural collapse becomes dominant. For lower intensity levels, corresponding to return periods of 98 and 224 years, the median repair costs (i.e., at 50% probability of non-exceedance) are equal to 1,514,285 € and 4,966,666 €, respectively, corresponding to 15.14% and 49.66% of the total replacement cost. Both values remain below the threshold of 60% of the replacement cost. Beyond this threshold (approximately 6 million euros), as shown in Figure 15b, the total repair cost is assumed equal to the replacement cost of the building. As a consequence, the non-exceedance curve exhibits a plateau, reflecting the dominance of collapse scenarios. In particular, the structure exhibits a very low probability of non-exceedance (approximately 3%) at the highest intensity level (corresponding to a 5% probability of exceedance in 50 years), due to the high number of realizations leading to collapse. Conversely, for seismic events with return periods of 98, 224, and 475 years, the probabilities of non-exceedance of the maximum loss threshold are approximately 79.8%, 53.2%, and 19.2%, respectively.

A further decomposition of the results based on component fragility groups is presented in Figure 15c, highlighting the different contributions of each component type to the total repair cost. The reported percentages are referred to the total repair costs shown in Figure 15a, and therefore correspond to a 50% probability of non-exceedance.

Overall, the results indicate a relatively consistent contribution of most components across intensity levels, with the notable exception of external beam–column joints. In particular, when moving from the first intensity level (98 years) to the second (224 years), the economic contribution of external joints decreases, as higher damage states are progressively reached in external walls and, consequently, in columns, which begin to dominate the repair costs.

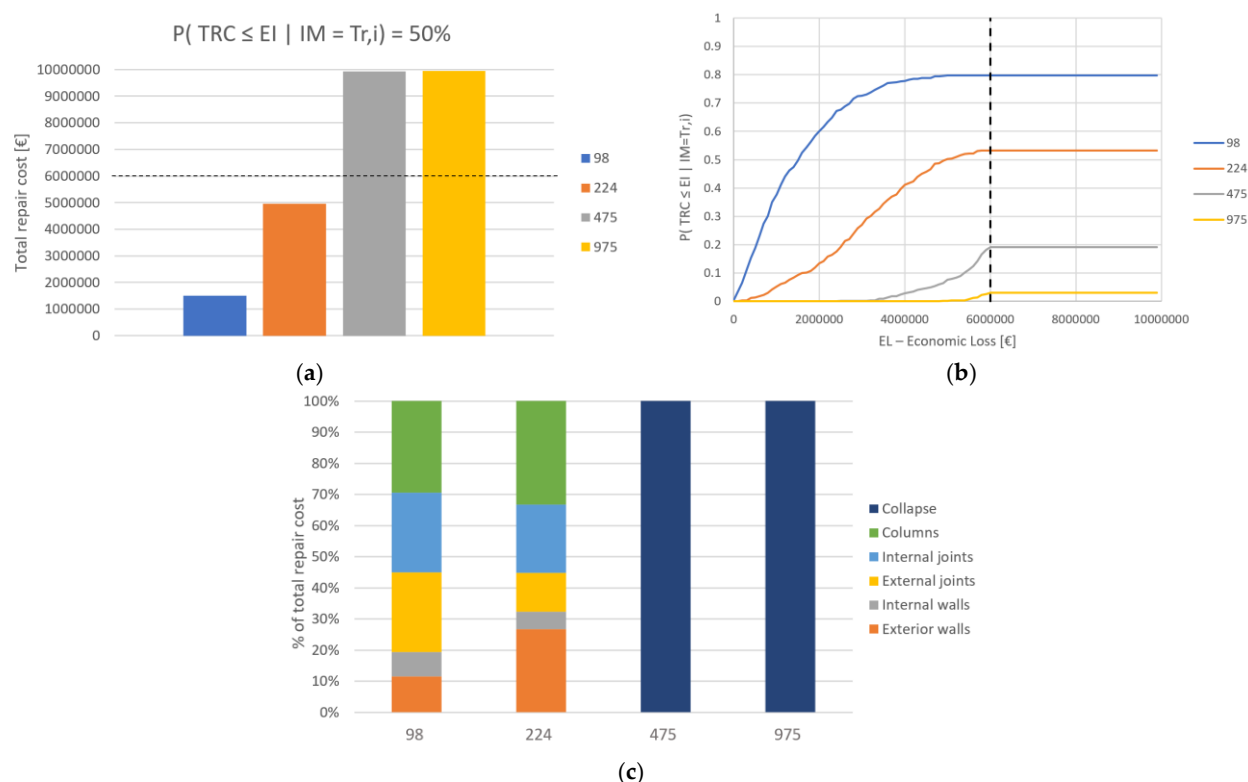


Figure 15. (a) Histograms of total repair cost (TRC) for the considered seismic intensity levels, corresponding to a 50% probability of non-exceedance; (b) probability of non-exceedance curves of total repair cost (TRC) as a function of economic loss (EL), highlighting the saturation at the replacement cost threshold; (c) normalized contribution of different structural component fragility groups (EWJ, IWJ, DWC) to the total repair cost at 50% probability of non-exceedance.

The ultimate objective of the analysis is to express the seismic risk of the building through the EAL, which represents the average annual economic loss associated with the seismic exposure and vulnerability of the structure. The probability of exceedance is combined with the corresponding mean annual frequency of occurrence, derived from the seismic hazard curve for each intensity interval. The resulting relationship is illustrated in Figure 16, where the area under the curve represents the Expected Annual Loss. This quantity provides an estimate of the mean economic loss that is expected to be incurred annually over the building's lifetime. The figure also highlights the relative contribution of each seismic intensity level to the overall loss. The computed value of the EAL is equal to 68,979.92 €, corresponding to approximately 0.7% of the total replacement cost of the building. This translates into an average annual repair cost of about 18 €/m². The primary purpose of a time-based economic assessment through the EAL metric is twofold. On one hand, it provides a valuable tool for insurance companies, supporting the definition of premiums for policies covering natural hazards. On the other hand, it offers a quantitative basis for cost-benefit analyses, enabling the comparison of different retrofit or strengthening strategies aimed at improving the seismic performance of the building. The obtained EAL corresponds to approximately 0.7% of the building replacement cost and falls within the range reported in previous PBEE studies on existing reinforced concrete buildings located in moderate-to-high seismicity regions. Although the value may appear limited when expressed on an annual basis, it represents the long-term economic consequences of seismic risk and therefore provides a meaningful indicator for decision-makers. In particular, the EAL can be directly used to compare alternative retrofit strategies, evaluate the economic efficiency of seismic strengthening interventions, and support insurance pricing schemes. For strategic facilities such as hospitals, where post-earthquake

functionality is essential, the EAL should be interpreted as a lower-bound estimate of the overall seismic impact, since indirect losses related to downtime and service interruption are not included in the present study.

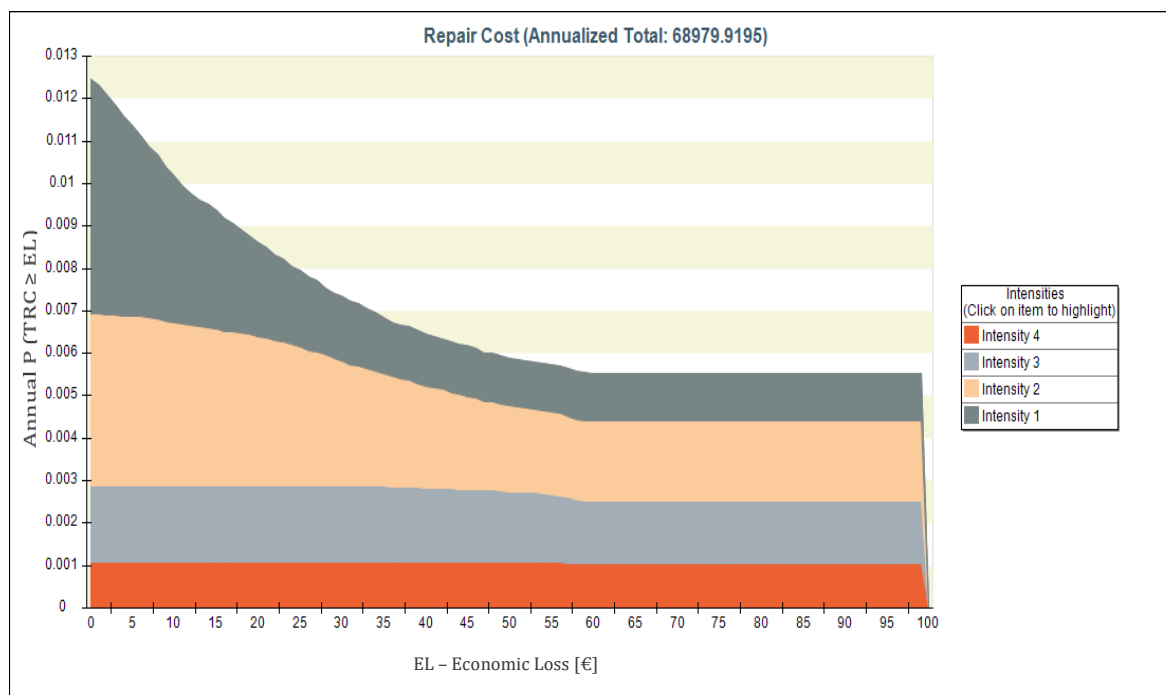


Figure 16. Cumulative distribution of the EAL for each seismic intensity level.

The results presented in Figures 13–16 provide a comprehensive picture of the economic consequences associated with the seismic vulnerability of the investigated building. While Figures 13 and 14 describe the probabilistic distribution of repair costs for each seismic intensity level, Figures 15 and 16 allow these results to be interpreted from a broader risk perspective. As expected, the repair costs increase significantly with increasing seismic intensity. For moderate seismic events (98- and 224-year return periods), the dominant contribution to the losses is associated with repairable structural and non-structural damage. In these cases, the median repair costs remain below the replacement threshold, indicating that the building is expected to remain economically repairable despite experiencing significant damage.

For higher seismic intensities, the probability of collapse progressively increases and becomes the dominant contributor to the overall losses. Consequently, the total repair cost tends to approach the replacement cost of the building, reflecting the transition from repair-controlled scenarios to collapse-controlled scenarios. This trend is particularly evident for return periods of 475 and 975 years, where collapse-related consequences govern a significant fraction of the simulated realizations.

The component-level breakdown shown in Figure 15c further highlights that the contribution of the different fragility groups varies with increasing seismic intensity. At lower intensity levels, repair costs are distributed among the various structural and non-structural components, whereas at higher intensity levels the contribution of the most vulnerable components becomes increasingly significant. This confirms the importance of accurately characterizing both structural and non-structural fragilities within a PBEE framework.

Finally, Figure 16 integrates the intensity-based results with the site-specific seismic hazard information and converts them into a time-based metric represented by the Expected Annual Loss (EAL). Unlike the repair costs associated with a single seismic scenario, the EAL accounts for both the probability of occurrence of seismic events and the associated economic consequences. Therefore, it provides a synthetic measure of the long-term seismic risk of the building. The obtained value of 68,979.92 € per year, corresponding to approximately 0.7% of the replacement cost, demonstrates how relatively infrequent but potentially severe seismic events can generate a measurable long-term economic burden over the service life of the structure.

From an engineering perspective, these results confirm that EAL can be interpreted as a global performance indicator capable of combining seismic hazard, structural vulnerability, collapse probability, and economic consequences into a single metric. Such information may support future risk-informed decisions concerning seismic retrofit prioritization, resilience assessment, insurance applications, and long-term asset management strategies.

6. Conclusions

This study presented a comprehensive performance-based seismic loss assessment of an existing reinforced concrete hospital building, representative of a large portion of the European building stock designed prior to modern seismic provisions. The adopted methodology integrates nonlinear structural analysis, component-based fragility and loss modeling, and site-specific seismic hazard characterization within the PBEE framework, ultimately leading to the evaluation of the EAL.

The results highlight that the seismic performance of the investigated structure is strongly influenced by shear-sensitive mechanisms, particularly at the level of beam–column joints and poorly confined columns. The nonlinear analyses demonstrated that local brittle failures can limit the global capacity of the structure and significantly affect its response, even at moderate intensity levels. This behavior was consistently observed in both pushover and nonlinear dynamic analyses, confirming the importance of accurately modeling shear-related degradation in existing buildings.

From an economic perspective, the study showed that repair costs increase rapidly with seismic intensity and tend to saturate at the replacement cost threshold when collapse becomes the governing condition. The probabilistic loss assessment revealed that, for moderate return periods, repair costs remain below critical thresholds, while for higher intensity levels the probability of collapse dominates the loss distribution.

The computed EAL value, equal to approximately 0.7% of the total replacement cost, provides a synthetic yet comprehensive indicator of the seismic risk associated with the building. This metric proves particularly effective in translating complex structural behavior into a quantitative economic measure, enabling a direct comparison between different design strategies or retrofit ones.

From a quantitative perspective, the results showed that the median repair costs corresponding to return periods of 98 and 224 years are approximately 1.51 M€ and 4.97 M€, respectively, corresponding to about 15% and 50% of the building replacement cost. For higher intensity levels, collapse progressively becomes the governing loss mechanism, causing the expected losses to approach the replacement cost threshold. The resulting Expected Annual Loss (EAL) was estimated as 68,979.92 €, corresponding to approximately 0.7% of the total replacement cost and an average annual repair cost of about 18 €/m². These values provide a quantitative measure of the long-term economic consequences associated with the seismic vulnerability of the investigated building.

The proposed approach offers significant potential applications in risk-informed decision-making, including cost–benefit analyses of seismic retrofitting interventions and

the definition of insurance premiums for natural hazard coverage. In particular, the EAL metric allows stakeholders to move from a purely safety-based perspective toward an integrated evaluation that accounts for both structural performance and economic consequences.

Future developments of this research may include the extension of the methodology to a broader range of building typologies, the incorporation of more detailed non-structural component modeling, and the use of larger ground motion datasets representative of different seismic regions. Additionally, the integration of indirect losses, such as downtime and loss of functionality, especially relevant for strategic buildings such as hospitals, would further enhance the robustness and applicability of the proposed framework. Furthermore, the adopted classification represents a simplified but commonly used approach for replacement cost estimation within performance-based loss assessments. Although more detailed room-by-room cost classifications are available in some national databases, the selected level of aggregation was considered adequate for the objectives and available information of the present study. Furthermore, future developments may benefit from the integration of Artificial Intelligence (AI) techniques with PBEE-based frameworks, enabling more efficient fragility assessment, surrogate modeling of structural response, rapid loss estimation, and risk-informed decision-making for large portfolios of existing buildings and critical infrastructures.

Some limitations of the present study should also be acknowledged. The nonlinear analyses were performed considering only the horizontal components of seismic excitation, consistently with the primary objective of evaluating the dominant sources of seismic demand affecting the investigated building. However, recent earthquake events have highlighted the potential importance of vertical ground-motion components, particularly for acceleration-sensitive non-structural elements, equipment, beam–column joints, and gravity-load-resisting members. Future research should therefore investigate the influence of vertical seismic excitation within the proposed PBEE framework and evaluate its effects on damage occurrence, economic losses, and Expected Annual Loss estimates for strategic facilities such as hospitals.

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References

1. Feliciano, D.; Arroyo, O.; Cabrera, T.; Contreras, D.; Valcarcel Torres, J.A.; Gomez Zapata, J.C. Seismic risk scenarios for the residential buildings in the Sabana Centro province in Colombia. *Nat. Hazards Earth Syst. Sci.* **2023**, *23*, 1863–1890. <https://doi.org/10.5194/nhess-23-1863-2023>.
2. Sousa, R.; Silva, V.; Rodrigues, H. The importance of indirect losses in the seismic risk assessment of industrial buildings—An application to precast RC buildings in Portugal. *Int. J. Disaster Risk Reduct.* **2022**, *74*, 102949. <https://doi.org/10.1016/j.ijdrr.2022.102949>.
3. Jaiswal, K.; Wald, D.J. Estimating Economic Losses from Earthquakes Using an Empirical Approach. *Earthq. Spectra* **2013**, *29*, 309–324. <https://doi.org/10.1193/1.4000104>.
4. León, J.A.; Ordaz, M.; Haddad, E.; Araújo, I.F. Risk caused by the propagation of earthquake losses through the economy. *Nat. Commun.* **2022**, *13*, 2908. <https://doi.org/10.1038/s41467-022-30504-3>.

5. Silva, V.; Crowley, H.; Jaiswal, K.S.; Acevedo, A.B.; Pittore, M.; Journey, M. Developing a global earthquake risk model. In Proceedings of the 16th European Conference on Earthquake Engineering, Thessaloniki, Greece, 18–21 June 2018; p. 11834.
6. Nahoaj, S. From Catastrophe to Recovery: The Impact of Natural Disasters on Economic Growth in Developed and Developing Countries. *Eur. J. Dev. Stud.* **2023**, *3*, 17–22 <https://doi.org/10.24018/ejdevelop.2023.3.2.237>.
7. Benson, C.; Clay, E.J. Disasters, Vulnerability and the Global Economy: Implications for Less-Developed Countries and Poor Populations. In *Developmental Entrepreneurship: Adversity, Risk, and Isolation*; Galbraith, C.S., Stiles, C.H., Eds.; United Nations Development Programme (UNDP): New York, NY, USA, 2006. [https://doi.org/10.1016/S1074-7877\(06\)05007-0](https://doi.org/10.1016/S1074-7877(06)05007-0).
8. Wouter Botzen, W.J.; Deschenes, O.; Sanders, M. The Economic Impacts of Natural Disasters: A Review of Models and Empirical Studies. *Rev. Environ. Econ. Policy* **2019**, *13*, 167–188. <https://doi.org/10.1093/reep/rez004>.
9. Hofer, L.; Zanini, M.A. Damage Scenario and Economic Losses Estimation of Historical Earthquakes Occurred in Northeastern Italy. In *Proceedings of the 1st Conference of the European Association on Quality Control of Bridges and Structures*; Pellegrino, C., Faleschini, F., Zanini, M.A., Matos, J.C., Casas, J.R., Strauss, A., Eds.; Lecture Notes in Civil Engineering; Springer: Cham, Switzerland, 2021; Volume 200. https://doi.org/10.1007/978-3-030-91877-4_84.
10. Di Pasquale, D.; Orsini, G. A Probabilistic Model for the Assessment of the Earthquake Economic Losses in Italy. In *Proceedings of the WIT Transactions on Ecology and the Environment*; WIT Press: Southampton, UK, 1998; Volume 31, p. 10.
11. Meroni, F.; Squarcina, T.; Pessina, V.; Locati, M.; Modica, M.; Zoboli, R. A Damage Scenario for the 2012 Northern Italy Earthquakes and Estimation of the Economic Losses to Residential Buildings. *Int. J. Disaster Risk Sci.* **2017**, *8*, 326–341. <https://doi.org/10.1007/s13753-017-0142-9>.
12. Hofer, L.; Zanini, M.A.; Faleschini, F.; Pellegrino, C. Expected losses vs earthquake magnitude curves, for seismic risk mitigation and for insurance purposes. *Procedia Struct. Integr.* **2023**, *44*, 1824–1831. <https://doi.org/10.1016/j.prostr.2023.01.233>.
13. Nobile, L. An Overview of Seismic Risk Management for Italian Architectural Heritage. *Struct. Durab. Health Monit.* **2023**, *17*, 353–368. <https://doi.org/10.32604/sdhm.2023.028247>.
14. Nutter, F.W. Financing Catastrophe Risk with Public and Private (Re)insurance Resources. In *Seeds of Disaster, Roots of Response: How Private Action Can Reduce Public Vulnerability*; Auerswald, P.E., Branscomb, L.M., La Porte, T.M., Michel-Kerjan, E.O., Eds.; Cambridge University Press: Cambridge, UK, 2006; pp. 379–392.
15. Auerswald, P.; Branscomb, T.; Erwann, M. *Seeds of Disaster, Roots of Response: How Private Action Can Reduce Public Vulnerability*; Cambridge University Press: Cambridge, UK, 2006.
16. Perazzini, S.; Gnecco, G.S.; Pammolli, F. A Public-Private Insurance Model for Natural Risk Management: An Application to Seismic and Flood Risks on Residential Buildings in Italy. *arXiv* **2020**, arXiv:2006.05840.
17. Gollier, C. *Some Aspects of the Economics of Catastrophe Risk Insurance*; CESifo Working Paper Series 1409; CESifo: Munich, Germany, 2005.
18. Ito, H.; McCauley, R.N. A Disaster Under-(Re)Insurance Puzzle: Home Bias in Disaster Risk-Bearing. *IMF Econ Rev.* **2022**, *70*, 735–772. <https://doi.org/10.1057/s41308-022-00172-w>.
19. Froot, K. The Limited Financing of Catastrophe Risk: An Overview. NBER Working Paper No. w6025, 1997. Available online: https://papers.ssrn.com/sol3/papers.cfm?abstract_id=226435 (accessed on 28 April 2026).
20. Manganeli, B.; Vona, M.; De Paola, P. Evaluating the cost and benefits of earthquake protection of buildings. *J. Eur. Real Estate Res.* **2018**, *11*, 263–278. <https://doi.org/10.1108/JERER-09-2017-0029>.
21. Zanini, M.A.; Hofer, L.; Pellegrino, C. A framework for assessing the seismic risk map of Italy and developing a sustainable risk reduction program. *Int. J. Disaster Risk Reduct.* **2019**, *33*, 74–93. <https://doi.org/10.1016/j.ijdr.2018.09.012>.
22. D’Alpaos, C.; Bragolusi, P. The Market Price Premium for Buildings Seismic Retrofitting. *Sustainability* **2020**, *12*, 8791. <https://doi.org/10.3390/su12218791>.
23. Gizzi, F.T.; Potenza, M.R.; Zotta, C. The Insurance Market of Natural Hazards for Residential Properties in Italy. *Open J. Earthq. Res.* **2016**, *5*, 35–61. <https://doi.org/10.4236/ojer.2016.51004>.
24. De Masi, F.; Porrini, D. Vulnerability to Natural Disasters and Insurance: Insights from the Italian Case. *Int. J. Financ. Stud.* **2018**, *6*, 56. <https://doi.org/10.3390/ijfs6020056>.
25. Mushonga, F.B.; Mishi, S. Natural hazard insurance demand: A systematic review. *Jamba J. Disaster Risk Stud.* **2022**, *14*, 1223. <https://doi.org/10.4102/jamba.v14i1.1223>.
26. Avcil, F. Investigation of Precast Reinforced Concrete Structures during the 6 February 2023 Türkiye Earthquakes. *Sustainability* **2023**, *15*, 14846. <https://doi.org/10.3390/su152014846>.
27. Nascimbene, R. Investigation of seismic damage to existing buildings by using remotely observed images. *Eng. Fail. Anal.* **2024**, *161*, 108282.

28. Işık, E.; Avcil, F.; Hadzima-Nyarko, M.; İzol, R.; Büyüksaraç, A.; Arkan, E.; Radu, D.; Özcan, Z. Seismic Performance and Failure Mechanisms of Reinforced Concrete Structures Subject to the Earthquakes in Türkiye. *Sustainability* **2024**, *16*, 6473. <https://doi.org/10.3390/su16156473>.
29. Kocakaplan Sezgin, S.; Sakcalı, G.B.; Özen, S.; Yıldırım, E.; Avcı, E.; Bayhan, B.; Çağlar, N. Reconnaissance report on damage caused by the February 6, 2023, Kahramanmaraş Earthquakes in reinforced-concrete structures. *J. Build. Eng.* **2024**, *89*, 109200.
30. Işık, E.; Avcil, F.; İzol, R.; Büyüksaraç, A.; Bilgin, H.; Harirchian, E.; Arkan, E. Field Reconnaissance and Earthquake Vulnerability of the RC Buildings in Adıyaman during 2023 Türkiye Earthquakes. *Appl. Sci.* **2024**, *14*, 2860.
31. Marinković, M.; Baballëku, M.; Isufi, B.; Blagojević, N.; Milićević, I.; Brzev, S. Performance of RC cast-in-place buildings during the 26 November 2019 Albania earthquake. *Bull. Earthq. Eng.* **2022**, *20*, 5427–5480.
32. Nascimbene, R.; Brunesi, E.; Sisti, A. Expeditious numerical capacity assessment in precast structures via inelastic performance-based spectra. *Heliyon* **2024**, *10*, e39729.
33. Magliulo, G.; Ercolino, M.; Petrone, C.; Coppola, O.; Manfredi, G. The Emilia earthquake: Seismic performance of precast reinforced concrete buildings. *Earthq. Spectra* **2014**, *30*, 891–912.
34. Bosio, M.; Di Salvatore, C.; Bellotti, D.; Capacci, L.; Belleri, A.; Piccolo, V.; Cavalieri, F.; Lago, B.D.; Riva, P.; Magliulo, G.; et al. Modelling and Seismic Response Analysis of Non-residential Single-storey Existing Precast Buildings in Italy. *J. Earthq. Eng.* **2023**, *27*, 1047–1068. <https://doi.org/10.1080/13632469.2022.2033364>.
35. Furtado, A.; Rodrigues, H.; Arêde, A.; Varum, H. A review of the performance of infilled rc structures in recent earthquakes. *Appl. Sci.* **2021**, *11*, 5889.
36. Pavese, A.; Lanese, I.; Nascimbene, R. Seismic Vulnerability Assessment of an Infilled Reinforced Concrete Frame Structure Designed for Gravity Loads. *J. Earthq. Eng.* **2017**, *21*, 267–289.
37. Brunesi, E.; Nascimbene, R.; Pavese, A. Mechanical model for seismic response assessment of lightly reinforced concrete walls. *Earthq. Struct.* **2016**, *11*, 461–481.
38. Georgiev, T. Field Observations and Assessment of Performance Level of Steel Structures After the Kahramanmaraş Earthquake Sequence of 2023. In Proceedings of the 11th International Conference on Behaviour of Steel Structures in Seismic Areas, Salerno, Italy, 10 July 2024.
39. Iyama, J.; Matsuo, S.; Kishiki, S.; Ishida, T.; Azuma, K.; Kido, M.; Iwashita, T.; Sawada, K.; Yamada, S.; Seike, T. Outline of reconnaissance of damaged steel school buildings due to the 2016 Kumamoto earthquake. *AII J. Technol. Des.* **2018**, *24*, 183–188.
40. Matsumoto, Y.; Koyama, T.; Yamada, S.; Iyama, J.; Kishiki, S.; Shimada, Y. Seismic damage to roof and non-structural components in steel school buildings due to the 2011 tohoku earthquake. *AII J. Technol. Des.* **2014**, *20*, 121–126.
41. DMLP. Decreto del Ministro dei Lavori Pubblici. Norme Tecniche per la Progettazione, Esecuzione e Collaudo Degli Edifici in Muratura e per il Loro Consolidamento. Gazzetta Ufficiale Della Repubblica Italiana 5 December 1987 n. 285. Available online: <http://www.daddio.it/Norme/dm201187.pdf> (accessed on 12 April 2026). (In Italian)
42. DMLP. Decreto del Ministro dei Lavori Pubblici del 16 Gennaio 1996. Norme Tecniche per le Costruzioni in Zone Sismiche. Gazzetta Ufficiale Della Repubblica Italiana, 5 February 1996, n. 19 Available online: https://www.staticaesismica.it/normative/DM_16_01_1996.pdf (accessed on 12 April 2026). (In Italian)
43. Merino, R.J.; Perrone, D.; Nascimbene, R.; Filiatrault, A. Performance-based seismic classification of acceleration-sensitive non-structural elements. *Earthq. Eng. Struct. Dyn.* **2023**, *52*, 4222–4244. <https://doi.org/10.1002/eqe.3973>.
44. Filiatrault, A.; Sullivan, T. Performance-based seismic design of nonstructural building components: The next frontier of earthquake engineering. *Earthq. Eng. Eng. Vib.* **2014**, *13*, 17–46. <https://doi.org/10.1007/s11803-014-0238-9>.
45. Rota, M.; Zito, M.; Dubini, P.; Nascimbene, R. On the Use of Accelerometric Data to Monitor the Seismic Performance of Non-Structural Elements in Existing Buildings: A Case Study. *Buildings* **2023**, *13*, 2651. <https://doi.org/10.3390/buildings13102651>.
46. Baird, A.; Tasligedik, A.S.; Palermo, A.; Pampanin, S. Seismic performance of vertical nonstructural components in the 22 February 2011 Christchurch earthquake. *Earthq. Spectra* **2014**, *30*, 401–425. <https://doi.org/10.1193/031013EQS067M>.
47. Zerbe, R.O.; Falit-Baiamonte, A. *The Use of Benefit-Cost Analysis for Evaluation of Performance-Based Earthquake Engineering Decisions*; PEER Report 2002/06; Pacific Earthquake Engineering Research Center, United States-Japan Cooperative Research Program: Berkeley, CA, USA; Pacific Earthquake Engineering Research Center: Berkeley, CA, USA, 2002.
48. Ahmed, H.A.; Tanoli, W.A. Seismic Retrofitting of RC Buildings Using a Performance-Based Approach for Risk Resilience and Vulnerability Assessment. *Buildings* **2025**, *15*, 1333. <https://doi.org/10.3390/buildings15081333>.
49. Ahmed, H.A.; Li, Y. Performance-based seismic retrofitting and resilience assessment for RC buildings considering direct and indirect losses. *Adv. Struct. Eng.* **2025**, *28*, 2237–2259. <https://doi.org/10.1177/13694332251325919>.

50. Goulet, C.A.; Haselton, C.B.; Mitrani-Reiser, J.; Beck, J.L.; Deierlein, G.G.; Porter, K.A.; Stewart, J.P. Evaluation of the seismic performance of a code-conforming reinforced-concrete frame building—From seismic hazard to collapse safety and economic losses. *Earthq. Engng. Struct. Dyn.* **2007**, *36*, 1973–1997. <https://doi.org/10.1002/eqe.694>.
51. Bianchi, S.; Ciurlanti, J.; Overend, M.; Pampanin, S. A probabilistic-based framework for the integrated assessment of seismic and energy economic losses of buildings. *Eng. Struct.* **2022**, *269*, 114852. <https://doi.org/10.1016/j.engstruct.2022.114852>.
52. Cavalieri, F.; Bellotti, D.; Caruso, M.; Nascimbene, R. Comparative evaluation of seismic performance and environmental impact of traditional and dissipation-based retrofitting solutions for precast structures. *J. Build. Eng.* **2023**, *79*, 107918. <https://doi.org/10.1016/j.jobbe.2023.107918>.
53. Iervolino, I.; Baraschino, R.; Spillatura, A. Evolution of Seismic Reliability of Code-Conforming Italian Buildings. *J. Earthq. Eng.* **2023**, *27*, 1740–1768. <https://doi.org/10.1080/13632469.2022.2087801>.
54. Federal Emergency Management Agency (FEMA). *Seismic Performance Assessment of Buildings, Volume 1—Methodology*; Report No. FEMA P-58-1; Federal Emergency Management Agency (FEMA): Washington, DC, USA, 2012.
55. Verki, A.; Aval, S. Performance-Based Design through Implementation of FEMA P-58 Methodology in Developing Countries. *Open J. Earthq. Res.* **2020**, *9*, 255–272. <https://doi.org/10.4236/ojer.2020.93015>.
56. Cardone, D.; Perrone, G. Damage and Loss Assessment of Pre-70 RC Frame Buildings with FEMA P-58. *J. Earthq. Eng.* **2017**, *21*, 23–61. <https://doi.org/10.1080/13632469.2016.1149893>.
57. Manafpour, A.; Moghaddam, P.K. Probabilistic Approach to Performance-Based Seismic Design of RC Frames. In *Vulnerability, Uncertainty, and Risk*; American Society of Civil Engineers (ASCE): Reston, Virginia, 2014; pp. 1736–1745. <https://doi.org/10.1061/9780784413609.174>.
58. Beilic, D.; Casotto, C.; Nascimbene, R.; Cicola, D.; Rodrigues, D. Seismic fragility curves of single storey RC precast structures by comparing different Italian codes. *Earthq. Struct.* **2017**, *12*, 359–374. <https://doi.org/10.12989/eas.2017.12.3.359>.
59. Rota, M.; Zito, M.; Bianchi, F.; Dubini, P. A New Seismic Classification Procedure for Nonstructural Elements Based on Fragility Curves. *Buildings* **2023**, *13*, 1017. <https://doi.org/10.3390/buildings13041017>.
60. Yang, T.Y. Assessing seismic risks for new and existing buildings using performance-based earthquake engineering (PBEE) methodology. In *Woodhead Publishing Series in Civil and Structural Engineering, Handbook of Seismic Risk Analysis and Management of Civil Infrastructure Systems*; Tesfamariam, S.; Goda, K., Eds.; Woodhead Publishing: Cambridge, UK, 2013; pp. 307–333. <https://doi.org/10.1533/9780857098986.3.307>.
61. Mahoney, M.; Hanson, B. *PACT-Performance Assessment Calculation Tool User Guide “ATC-58 Guidelines for Seismic Performance Assessment of Buildings”*; Department Of Homeland Security Federal Emergency Management Agency: Washington, DC, USA, 2007.
62. Kahandawa, K.A.R.V.D.; Domingo, N.D.; Park, K.S.; Uma, S.R. Earthquake damage estimation systems: Literature review. *Procedia Eng.* **2018**, *212*, 622–628. <https://doi.org/10.1016/j.proeng.2018.01.080>.
63. Sullivan, T.J. Use of Limit State Loss versus Intensity MoThe dels for Simplified Estimation of Expected Annual Loss. *J. Earthq. Eng.* **2016**, *20*, 954–974. <https://doi.org/10.1080/13632469.2015.1112325>.
64. Perrone, G.; Cardone, D.; O’Reilly, G.J.; Sullivan, T.J. Developing a Direct Approach for Estimating Expected Annual Losses of Italian Buildings. *J. Earthq. Eng.* **2022**, *26*, 1–32. <https://doi.org/10.1080/13632469.2019.1657988>.
65. McKenna, F. *OpenSees: Open System for Earthquake Engineering Simulation*; Pacific Earthquake Engineering Research Center, University of California: Berkeley, CA, USA, 2011. Available online: <https://opensees.berkeley.edu> (accessed on 28 April 2026).
66. Spacone, E.; Filippou, F.C.; Taucer, F.F. Fibre Beam–Column Model For Non-Linear Analysis Of R/C Frames: Part I. Formulation. *Earthq. Engng. Struct. Dyn.* **1996**, *25*, 711–725. [https://doi.org/10.1002/\(SICI\)1096-9845\(199607\)25:7<711::AID-EQE576>3.0.CO;2-9](https://doi.org/10.1002/(SICI)1096-9845(199607)25:7<711::AID-EQE576>3.0.CO;2-9).
67. Rajapakse, R.M.C.M.; Wijesundara, K.K.; Nascimbene, R.; Bandara, C.S.; Dissanayake, R. Accounting axial-moment-shear interaction for force-based fiber modeling of RC frames. *Engng Struct* **2019**, *184*, 15–36.
68. Hippola, H.M.S.S.; Wijesundara, K.K.; Nascimbene, R. Response of shear critical reinforced concrete frames and walls under monotonic loading. *Eng. Struct.* **2022**, *251*, 113483. <https://doi.org/10.1016/j.engstruct.2021.113483>.
69. Rasulo, A.; Pelle, A.; Lavorato, D.; Fiorentino, G.; Nuti, C.; Briseghella, B. Finite Element Analysis of Reinforced Concrete Bridge Piers Including a Flexure-Shear Interaction Model. *Appl. Sci.* **2020**, *10*, 2209. <https://doi.org/10.3390/app10072209>.
70. Ceresa, P.; Petrini, L.; Pinho, R.; Sousa, R. A fibre flexure–shear model for seismic analysis of RC-framed structures. *Earthq. Engng. Struct. Dyn.* **2009**, *38*, 565–586. <https://doi.org/10.1002/eqe.894>.

71. Liu, J.; Huang, L.; Wu, Z.; Tomlinson, D.; Cruz-Noguez, C.; Alexander, J.; Adeeb, S.; Li, Y. Development of enhanced fiber beam element with multi-axial material constitutive models for reinforced/prestressed concrete beams. *Eng. Struct.* **2021**, *248*, 113289. <https://doi.org/10.1016/j.engstruct.2021.113289>.
72. Sezen, H.; Moehle, J.P. Shear Strength Model for Lightly Reinforced Concrete Columns. *J. Struct. Eng.* **2004**, *130*, 1692–1703. [https://doi.org/10.1061/\(asce\)0733-9445\(2004\)130:11\(1692\)](https://doi.org/10.1061/(asce)0733-9445(2004)130:11(1692)).
73. Wibowo, A. Shear Capacity of Lightly Reinforced Concrete Columns. In *Proceedings of the ICCOEE2020*; Mohammed, B.S., Shafiq, N., Rahman, M. Kuty, S., Mohamad, H., Balogun, A.L., Eds.; Lecture Notes in Civil Engineering; Springer: Singapore, 2021; Volume 132. https://doi.org/10.1007/978-981-33-6311-3_50.
74. Zhang, J. Diagonal cracking and shear strength of reinforced concrete beams. *Mag. Concr. Res.* **1997**, *49*, 55–65. <https://doi.org/10.1680/mac.1997.49.178.55>.
75. ACI-ASCE Committee 426. The Shear Strength of Reinforced Concrete Members. *J. Am. Concr. Inst.* **1973**, *70*, 407–434.
76. ACI Committee 318; Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02). American Concrete Institute: Farmington Hills, MI, USA, 2002.
77. Günay, S.; Mosalam, K.M. PEER Performance-Based Earthquake Engineering Methodology, Revisited. *J. Earthq. Eng.* **2013**, *17*, 829–858. <https://doi.org/10.1080/13632469.2013.787377>.
78. Arroyo, O.; Liel, A.; Gutiérrez, S. A Performance-Based Evaluation of a Seismic Design Method for Reinforced Concrete Frames. *J. Earthq. Eng.* **2018**, *22*, 1900–1917.
79. NTC2018; Norme Tecniche per le Costruzioni—Decreto Ministeriale 17 Gennaio 2018. Ministry of Infrastructure and Transport: Roma, Italia, 2018. (In Italian)
80. EN 1998-1-1:2024; Eurocode 8: Design of Structures for Earthquake Resistance—Part 1: General Rules, Seismic Actions and Rules for Buildings. European Commission: Bruxelles, Belgium.
81. Palanci, M.; Kalkan, A.; Senel, S.M. Investigation of shear effects on the capacity and demand estimation of RC buildings, *Struct. Eng. Mech.* **2016**, *60*, 1021–1038. <https://doi.org/10.12989/sem.2016.60.6.1021>.
82. Increased Resilience of Critical Infrastructure Under Natural and Human-Induced Hazards (INFRA-NAT). 2019. Available online: <https://civil-protection-humanitarian-aid.ec.europa.eu> (accessed on 2 May 2026).
83. Chalarca, B.; Perrone, D.; Filiatrault, A.; Nascimbene, R.; Blasi, G.; Aiello, M.A. Braceless Seismic Restraint for Suspended Non-structural Elements: Concept, Design, and Numerical and Experimental Assessment. *J. Earthq. Eng.* **2026**, *30*, 135–164. <https://doi.org/10.1080/13632469.2025.2497955>.
84. Demir, A.; Palanci, M.; Kayhan, A.H. Enhanced Conditional Ground Motion Selection Model Considering Spectral Compatibility and Variability of Three Components for Multi-Directional Analysis. *Appl. Sci.* **2025**, *15*, 4135. <https://doi.org/10.3390/app15084135>.
85. Sdino, L.; Brambilla, A.; Dell’Ovo, M.; Sdino, B.; Capolongo, S. Hospital Construction Cost Affecting Their Lifecycle: An Italian Overview. *Healthcare* **2021**, *9*, 888. <https://doi.org/10.3390/healthcare9070888>.
86. Berto, R.; Rosato, P.; Stival, C.A. *Technical and Economic Valuation of Healthcare Building Upgrade*; Springer Nature: Cham, Switzerland, 2022.
87. Vona, M.; Manganelli, B.; Tataranna, S.; Anelli, A. An Optimized Procedure to Estimate the Economic Seismic Losses of Existing Reinforced Concrete Buildings due to Seismic Damage. *Buildings* **2018**, *8*, 144. <https://doi.org/10.3390/buildings8100144>.

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