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The seismic vulnerability assessment methodologies: A state-of-the-art review

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ABSTRACT

In the past decades, the research and development of methodologies have received considerable attention which quantified earthquake-related damages to structures. Among these, indices of seismic risk and vulnerability assessment have indeed been developed to quantify the level of damages to structural elements or the whole structural system. In this paper, a detailed investigation has been done on the developed methodologies in the field, and the findings from other works are summarized. The authors have tried to present the most common empirical and analytical methodologies in a concise manner, which would motivate researchers and practicing engineers to use it as a comprehensive guide and reference for their future works.

1. Introduction

Human casualties and economic losses caused by natural disasters have been dramatically increased in the last couple of decades [1]. Among these natural disasters, earthquake has been the most catastrophic phenomena. According to CATDAT damaging earthquake database, the 2010 Haiti earthquake resulted in a death toll that was estimated to be in the range of 46,000–316,000 casualties. 12 months later in Japan, the Tohoku earthquake in 2011 caused 20,475 fatalities and left 1.108 million people homeless. Besides the population losses that occurred in the 2011 Tohoku earthquake, the economic loss was $140 billion. Furthermore, on the economic front, a financial loss caused in Turkey was $2.2 billion after the occurrence of Van earthquake event in 2011, and was estimated as $1.7 billion when Sikkim earthquake event struck India in 2011 [2]. Seismic assessment performance of current buildings and infrastructure attracted considerable attention of the seismologists, due to the susceptibility and the lack of performance of these structures in the world over the past decade. Therefore, the seismic vulnerability speculation of building structures has become a major concern via evolving seismic assessment procedures [3]. Generally, the assessment procedures of an individual building rely on different parameters. These parameters focus on the structural system, seismic capacity, ground conditions, plane and elevation regularity, and limited field data collections. These parameters provide an image or realistic estimation of the structural system behavior. In other words, the risks from a seismic hazard are the possibilities to reach some significant losses at a certain interval period. These losses are identified as an economic index that should be compensated back to the system for condition assessment before a seismic event occurs. The disparity in the structure and building safety due to earthquakes deteriorations is impliedly recognized through construction tagging approaches, that are implemented after the most and major important seismic activities [4,5]. In such approaches, post-earthquake protection is commonly analyzed via full visual inspection, including such professional evaluation of the level of damage, extent, and the associated constructing usability through a group of skilled specialists. Performing seismic evaluation and investigation are common requirements for any crisis management application. Hazard examination enables the prospective failure from seismic threat in a relief stage that will be decided and makes a difference to create emergency plans. However, a need for clear guidelines and measures for repairing after a harming seismic tremor has frequently been discussed.

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Nevertheless, if the wide-range vulnerability is taken into consideration inside a regular quantitative evaluation framework system, analytical modeling of constructing the overall performance loss (PL) is foremost and preferable [6,7]. Many methods for seismic risk assessment were proposed by researchers as part of loss prediction which was classified into two major groups; empirical and analytical methods. In empirical vulnerability methods, the scale of damage was used as an inquiry approach to develop the data of the post-event that come with statistics studies as the content of building damages, whereas analytically, which were based on the limit stages and the mechanical attributes or quality of the structures. The purpose of this review paper is to investigate the available seismic vulnerability assessment methods in literature with a focus on the empirical and analytical seismic vulnerability indices.

2. Empirical assessment approach

2.1. Rapid Visual screening assessment methods

There are several rapid assessment methods such as the street screening method. Street screening method as a procedure is the simplest rapid assessment approach. Rapid Visual Screening (RVS) as a qualitative estimation procedure can be used on a large building stock to classify the vulnerability of the structures. It is built on observations made from the building exterior, without taking into consideration the building inside. This visual survey can be done in less than 30 min [8]. The FEMA standards in the U.S developed several guidelines for the risk assessment and retrofitting of structural buildings such as FEMA 310 [9]. However, based on FEMA 154 [10] the street screening method is known as the Rapid Visual Screening Method. This method is the first step in the assessment before going into a detailed assessment procedure and classifying the buildings according to their construction materials and their structural systems. Basically, it is a sidewalk survey technique that worked on detecting and observing building parameters and calculating the basic structural performance score for determining the risk priorities for buildings. The process starts, with the performance score that was calculated based on the building features, such as in FEMA 154. There are 17 buildings types introduced for the RVS procedure and for each type, a Basic Structural Hazard (BSH) score was determined. The BSH score is about the probability of collapse for a building structure. The final score was expressed as a negative of the logarithm (Base 10) as represented in Eq. (1). For example, if the final score would be 2, this signifies the probability of (10^{-2}) which is equal to 1% damage. After that, the BSH was modified by adding or subtracting the score modifiers (SMs) of a building as shown in Eq. (2). The score modifiers were based on the building properties that are affected by the seismic performance such as the number of stories, height, plan irregularity, vertical irregularity, the age of the buildings, and soil types. A building with a final score of less than 2 should undergo a more detailed investigation.

$$BSH = -\log10[P(colapse)] \tag{1}$$

$$S = BSH \pm SMs \tag{2}$$

Wallace and Miller [11], have applied the RVS procedure suggested by FEMA 154 for 1075 buildings in western Oregon in the U.S. Implementing the RVS procedure, they identified the potential effect of seismic hazard to public facilities. Moreover, Holmes [12] investigated some of the buildings in the US that have poor seismic performance due to an inadequate seismic design by using rapid screening techniques. Meanwhile, RVS strategy was developed in numerous other nations. A few of these RVS strategies are; Canada, Japan, Turkish, Greece, New Zealand, and Indian. In Canada, the National Research Council (NRC) has proposed the widely used seismic screening procedure [13]. The purpose of this method was to establish the Seismic Priority Index (SPI) resulting from the addition of the structural (SI) and non-structural (NSI) indices as shown in Eq. (3). This screening score major factors have been; building location, soil type, duration or age of occupancy, falling hazard, and others. The SPI index is categorized into three evaluation stages, where SPI less than 10 is considered as "low" detailing assessment, for SPI between 10 and 20, it is considered as "medium", and for SPI higher than 20, it is considered as "high" assessment [14,15].

$$SPI = SI + NSI \tag{3}$$

where SI is the structural index developed based on the product of five parameters. These parameters are; (A), Seismicity Index; (B), Effect of Soil Condition; (C), Type of Structure; (D), Building Irregularities; (E), Building Importance. The NSI is the non-structural index, which is the product of three parameters (B), (E), and (F). Where; F is the maximum value between F1 for falling hazards to life and F2 for hazard to vital operations.

In Japan, the Japanese Seismic Index approach comes in the form of three screening assessment stages to perform. In the first stage, the compressive strengths of the vertical resisting members are used to quantify the structure's response behavior during lateral seismic loading. The second stage, the seismic capacity is evaluated by considering the dynamic properties of the resisting members only such as ductility and strength, while in the third stage, the vertical and the horizontal members (columns, walls, and beams) strength and ductility are included for evaluating the structural performance during the earthquake movements. The Index of the structure (I_s) is calculated based on the product of Basic Structural (E_o) to Irregularity Index (S_D), as well as the time or deterioration index (T) as shown in Eq. (4). Once the Seismic Performance Index (I_S) has been determined, it ought to be compared with the Seismic Judgment Index (I_{S0}) to classify the building as adequate or not to resist earthquake forces as represented in Eq. (5). There are two possibilities in comparing I_S and I_{S0}, in the first one, if $I_S > I_{S0,}$ this means it has low vulnerability condition, and for the second one, if $I_S < I_{S0}$ it will correspond to high vulnerability condition [16,17].

$$Is = Eo \times SD \times T \tag{4}$$

$$Iso = Es \times Z \times G \times U \tag{5}$$

where (E_s) was taken as 0.8 for the first level of assessment, and 0.6 for the second and third levels; (Z) is the zonation index which corresponds to the building location; (G) is symbolized to the ground index, and (U) to the usage index.

In Turkey, Hassan and Sozen [18] developed the Priority Index procedure for every individual building, which consisted of the column index (CI) defined as the ratio of column area to the floor area, and the wall index (WI) as the ratio of areas, between the area of shear and infill walls divided by the floor area. In addition, Yakut [19] proposed a methodology based on the material and size properties, lateral resisting system, elements orientation, vertical and plan irregularities, column length, and workmanship. From these parameters, the capacity index (CI) can be computed to classify the building risk vulnerability. Bal et al. [20] proposed the P25 Scoring Method, which tends to classify the collapse-vulnerable buildings. This method was developed based on collected data of 323 buildings that suffered different levels of damages during earthquake events. The P25 Scoring method depends on some parameters such as material quality, steel corrosion, vertical and horizontal irregularities, ground conditions, depth of foundation, seismicity, and others. Seven different scores for different failure modes, from P1 to P7, between 0 and 100 varied from worst to best, respectively. The Turkish methods are summarized in Table 1.

In New Zealand, the society for earthquake engineering in 2012 recommended two stages of assessment: Initial Evaluation Procedure (IEP), and a Detailed Seismic Assessment (DSA). To perform the %NBS value it needs data to be collected such as seismic zone, soil type, construction age, and the design date of the building. After producing the %NBS values, the assessment is completed. If the (%NBS $\leq$ 33), this implies that the building is ultimately susceptible and required a supplementary detailed and precise assessment. For %NBS of 67 or more, it means buildings are capable of resisting future earthquakes. For (33 < %NBS < 67) more evaluation may be required [21].

The previous RVS tools are rapid and useful for estimating building response due to earthquake loadings, but still have disadvantages and drawbacks based on the observed and watched damage information. These methodologies do not involve all the structural typologies as well as the seismic intensities, which are essential to be considered for vulnerability estimation. These methods were generally based on expert judgment and statistical data and are not very reliable.

2.2. Vulnerability index methods

2.2.1. GNDT approach

During the last decades, the vulnerability index methodologies have been developed in Italy by "The National Group of Defense from Earthquakes denoted by GNDT approach" and were classified into two levels [22]. The methodology of "GNDT level I" classified the typologies of the buildings and defined the vulnerability classes (A, B, and C). The methodology of "GNDT level II" was related to Benedetti et al. [23]; Terremoti [24]; Benedetti and Petrini [25] and GNDT1993 approach. In this approach, a large number of damages survey data and information needed to be collected. The field survey is to build up a clear vision to understand the most fundamental parameters that were influencing and controlling the structural vulnerability of the building. For instance, plan layout and its elevation configurations, footing type, material type, and quality. There were eleven parameters in total, and one of the qualification coefficients *Ki* or *Cvi*, was distributed into four vulnerability classes (A, B, C, and D) for each of them. Each parameter assessed one structural attribute that is related to the building response during seismic loading. Then the parameters were weighted by considering the importance of each one, from less significant vulnerability parameters to the foremost important, where the weight values depended on expert judgment and opinion. This detailed information was merged with coefficients to establish the vulnerability index (Iv) that classified building damage under the excitations of an earthquake. To estimate the global seismic vulnerability index for each individual or group of building structures, the following equation is used.

$$Iv = \sum_{i=1}^{i=11} \frac{K_i \times W_i}{382.5} \tag{6}$$

The eleven mentioned parameters are summarized in Table 2:

Table 1
Rapid Visual Screening (Turkish Method).

Method	Index	Equation
RVS	Priority Index	$PI = CI + WI$
RVS	Capacity Index	$CPI = C_A \times C_M \times B \times CPI$
RVS	Performance Score index	$P = \alpha \times \beta \times P_{\min}$

*CI: Column Index, WI: Wall Index, C_A and C_M are coefficients factors reflecting the architectural features, α and β are correction factors, Pmin: smallest performance score.

The vulnerability index range of variation was between 0 and 382.5 as shown in Table 2, but generally, the range is normalized from 0 to 100 by dividing the values obtained from the weighted sum with 3.825. Where 0 is the minimum value which signifies the least vulnerable building and 100 is the worst case which indicates as the most vulnerable building. The derived data from previous earthquakes events were utilized to express the vulnerability functions in relation to the vulnerability index (Iv) with respect to damage factor (d) of the buildings. The damage factor as a definition is the proportion of the restoration cost to the replacement cost. The damage scale is between (0 < d < 1) as shown in Fig. 1; where d is assumed negligible or vanished (d = 0) for peak ground acceleration (y_i), and it increases linearly till it leads to collapse with damage index (d = 1) of peak ground acceleration (y_c). The controlling parameters have been evaluated according to the following empirical equations in terms of vulnerability index (Iv):

$$Y_i = \alpha i \times \exp[-\beta i(Iv + 25)] \tag{7}$$

$$Y_c = \left[\alpha c + \beta c(Iv + 25)^{\gamma}\right]^{-1} \tag{8}$$

where αi= 0.155, βi = 0.0207, αc = 0.625, βc = 0.00029, and γ = 2.145. These values were determined for RC- and masonry structures, which were used in seismic risk study in "Catania Project", Italy by Faccioli et al. [26]. In addition, the following equation can be used to apply a logarithmic relationship between ground motion intensities (PGA) or (y) and the MCS (I_{MCS}) [27]:

$$Log_e(y) = a.I_{MCS} - b, \text{ where a} = 0.605, \text{and b} = 7.073 \tag{9}$$

In the 2nd level of the GNDT method, a similar relationship was applied for RC buildings, but the main difference was in the parameters' weights that were all assumed to be equal to 1.0. These parameters described the deficiencies and the faults of the structure depending on expert visual observations. Furthermore, a criterion to describe vulnerability classes from less vulnerable "A" to most vulnerable "C" is also proposed as shown in Table 3. Therefore, to compare the masonry as well as the RC buildings, vulnerability indices of RC structures Iv* can be transformed in equivalent to masonry vulnerability indices by using the following formulas (Eqs. (10) and (11)):

$$\text{if Iv}^* > -6.5 \rightarrow Iv = -10.07Iv^* + 2.5175 \tag{10}$$

$$\text{if Iv}^* < -6.5 \rightarrow Iv = -1.731Iv^* + 2.5175 \tag{11}$$

2.2.2. European Macro-Seismic (EMS) approach (RISK-UE)

Another approach that has been developed for the vulnerability assessment purpose in Europe is known as the RISK-UE project. This project is financed and supported by the European Union (EU). The primary goal of this project has been to integrate an overall seismic risk assessment methodology in European countries. This is due to the absence of a worldwide system developed in Europe, and thus the behavioral-economic and political effect of seismicity activity that happened in Tukey, Athens, and Greece. For this reason, the vulnerability index method (VIM) has therefore been introduced as a vulnerability assessment that was successfully created in seven European cities [28]. This approach is based on the building typology classification that is distributed into six vulnerability classes (A to F) from most vulnerable to least vulnerable typologies. Such buildings are classified into four general typologies: masonry, reinforced concrete, steel, and wooden. Besides that, it categorized the scale of damage into five grades denoted by D_1, D_2, D_3, D_4, D_5 from slightly damaged into fully collapsed [29]. The EMS-98 scale is planned to be used to clear up the

Table 2
Masonry building classes and relative weight of each parameter (GNDT, 1993).

Number	Parameters	Ki Classes				Weight
		A	B	C	D	Wi
1	Type and organization of resisting system	0	5	20	45	1.00
2	Resistant system quality	0	5	25	45	0.25
3	Aggregate strength	0	5	25	45	1.5
4	Location and foundation of building	0	5	15	45	0.75
5	Diaphragms horizontal elements	0	5	25	45	Variable
6	Configuration of plan layout	0	5	25	45	0.5
7	Configuration in height and elevation	0	5	25	45	Variable
8	Optimum distance between walls	0	5	25	45	0.25
9	Roof	0	5	25	45	Variable
10	Non-structural elements (NS)	0	5	25	45	0.25
11	Particular terms of maintenance	0	5	25	45	1.00

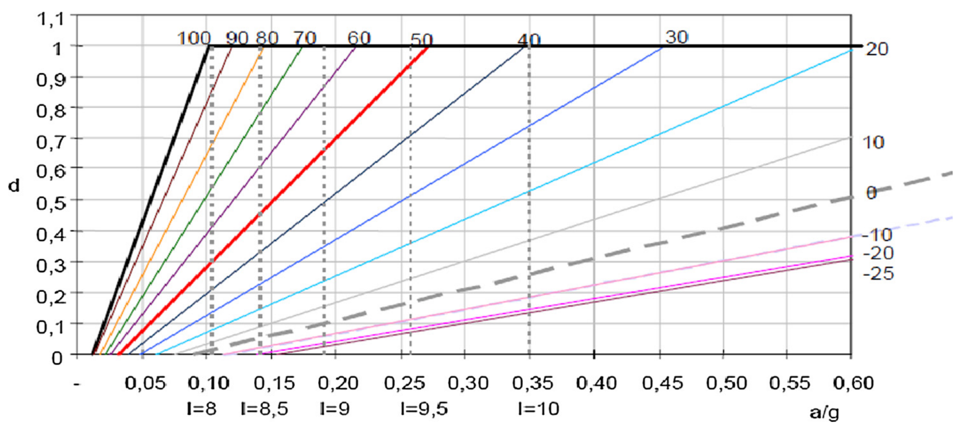

Fig. 1. Damage factor (d) vs. Ground motion intensities (PGA and MCS) with some vulnerability indices values (Iv) [25].

Table 3
RC buildings classes and relative weight of each parameter (GNDT, 1993).

Number	Parameters	Classes Cvi			Vulnerability Index
		A	B	C	
1	Type and organization of resisting system	0.00	−1.00	−2.00	$Iv^* = \sum_{i=1}^{i=11} Cvi$
2	Resistant system quality	0.00	−0.25	−0.50	
3	Aggregate strength	0.25	0.00	−0.25	
4	Location and foundation of building	0.00	−0.25	−0.50	
5	Diaphragms horizontal elements	0.00	−0.25	−0.50	
6	Configuration of plan layout	0.00	−0.25	−0.50	
7	Configuration in height and elevation	0.00	−0.50	−1.50	
8	Critical elements connections and links	0.00	−0.25	−0.50	
9	Elements of low ductility	0.00	−0.25	−0.50	
10	Non-structural elements (NS)	0.00	−0.25	−0.50	
11	Particular terms of maintenance	0.00	−0.50	−1.00	

vision and the definitions of many typology structures in the future for the European cities. As a methodology, it was established under the context of the RISK-UE project to take into account the current or the old constructions in the areas it covers [30]. This method measures the vulnerability of a single or set of structural buildings in terms of (V) by considering the typology features. The vulnerability index varies from the least vulnerable to the most vulnerable between 0 and 1. The values of the vulnerability indices are presented for each vulnerability class from A to F as a set of five values in Table 4 and Fig. 2. V_I* is the most tolerable value for each class of the vulnerability index (V_I). Where $V_I^{(--)}$, and $V_I^{(++)}$ are the top and bottom limits of the tolerable values, while $V_I^{(-)}$ and $V_I^{(+)}$ are the limits of the uncertainty range for V_I*. In this way, the typology vulnerability index (V_I*) values are practical in Europe that consisted of 15 building typologies as shown in Table 5.

Instead, the typological vulnerability index (V_I*) had to be modified based on some structural modifiers for reinforced concrete and masonry buildings. As highlighted before by the EMS scale, it is noted that the building's structural behavior depends on the structural system, but there are other factors that influence the building performance for example; construction quality, plan, and vertical irregularities, number of floors, foundations, and others. These modifiers are known as "The Behavior Modifier Factor/The Response Modification Factor" ΔV_m with a score symbolized as V_m. The modifying scores are attributed based on expert judgment. After some modifications, the total vulnerability index can be computed by adding or summing all the score modifiers as shown in the equation below:

$$\Delta V_m = \sum V_m \quad (12)$$

Table 4
Indices of the vulnerability for the six vulnerability classes [31].

Class	$V_I^{(--)}$	$V_I^{(-)}$	V_I^*	$V_I^{(+)}$	$V_I^{(++)}$
A	0.78	0.86	0.90	0.94	1.02
B	0.62	0.70	0.74	0.78	0.86
C	0.46	0.54	0.58	0.62	0.70
D	0.30	0.38	0.42	0.46	0.54
E	0.14	0.22	0.26	0.30	0.38
F	0.02	0.06	0.10	0.14	0.22

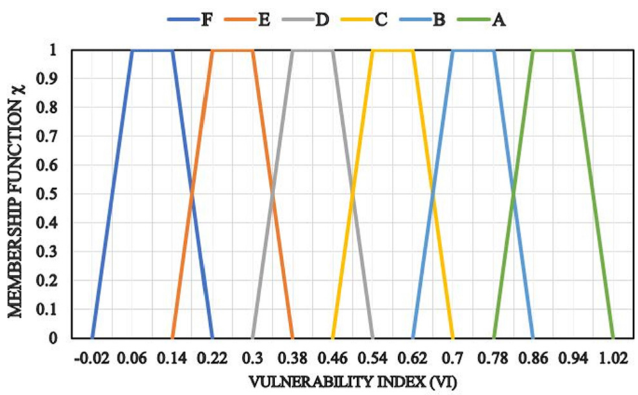

Fig. 2. Functions for six-class vulnerability index [30].

In addition to the "Behavior Modifier Factor", there is another modifier called the "Regional Vulnerability Factor" ΔV_R. This factor modifies the V_I^* based on the historical data and expert judgment. For example, in Lisbon, the ΔV_R equals to 0.12. Eventually, the total vulnerability index value may be calculated as follows:

$$VI = VI^* + \Delta V_m + \Delta V_R \tag{13}$$

In summary, Table 6 describes the way for determining the vulnerability index value for a single building implementing into the EMS approach.

2.2.3. Combined GNDT and macro-seismic approaches

Another type of vulnerability index is a combined approach. The first problem or the thing that should be dealt with is to find a correlation between these two methods. This can be proposed and expressed in terms of defining the damage grade (μ_D) as a vulnerability function. To apply this operational methodology for combining these two approaches, an analytical term was offered by Bernardini et al. [32] and other researchers (Giovinazzi and Lagomarsino [33]; Lantada et al. [34]; Azizi et al. [35]; Athmani et al. [36]; Maio et al. [37]; Ferreira et al. [38]; Athmani et al. [39]), which correlated the seismic hazards with respect to the mean damage grade (μ_D). This approach composed of six grading damages ($0 < \mu_D < 5$) in terms of masonry buildings and RC-buildings vulnerability. Thus, it permits to calculate the mean damage grade as can be seen in Table 7.

Table 6
Procedure for EMS vulnerability index [31].

Vulnerability Index Estimation for a Single building	
Typology V_I^*	Values from Table 5
$\mathbf{\Delta V_m}$	$\Delta V_m = \sum Vm$
$\mathbf{\Delta V_R}$	ΔV_R, Established based on expert judgment or previously observed damage data
Total Vulnerability Index	$VI = V_I^* + \Delta V_m + \Delta V_R$

$$f(V,I) = \begin{cases} 1 \rightarrow I > 7 \\ e^{\frac{V}{2}(I-7)} \rightarrow I \leqslant 7 \end{cases} \tag{14}$$

where (I) represents the earthquake, hazard associated with macroseismic intensity, (V) is the vulnerability index, (Q) identifies the ductility of a particular construction, ranging from 1 to 4.

In order to form a precise convergence between the two approaches, it is fundamental to fasten the damage factor adopted within the GNDT level II approach as represented previously in Fig. 1 to the physical damage grades characterized in the Macroseismic approach. This can be shown after getting the mean damage grade μ_D for every building, and then the economic damage indicator was found by utilizing the relationship proposed by FEMA-NIBS (Federal Emergency Management Agency). To express

Table 5
Indices of the vulnerability of building typologies [30].

	Building Type	Vulnerability Classes				
		$V_I^{(--)}$	$V_I^{(-)}$	V_I^*	$V_I^{(+)}$	$V_I^{(++)}$
Masonry Buildings	Rubble stone	0.62	0.810	0.873	0.980	1.02
	Adobe bricks	0.62	0.687	0.840	0.980	1.02
	Simple stone	0.46	0.650	0.740	0.830	1.03
	Massive stone	0.30	0.490	0.616	0.793	0.86
	URM (old bricks)	0.46	0.650	0.740	0.830	1.02
	URM + RC-Slabs	0.30	0.490	0.616	0.790	0.86
	Confined Masonry	0.14	0.330	0.451	0.633	0.70
RC Buildings	RC-Frame (No ERD)	0.30	0.490	0.644	0.800	1.02
	RC-Frame (M-ERD)	0.14	0.330	0.484	0.640	0.86
	RC-Frame (H-ERD)	−0.02	0.170	0.324	0.480	0.70
	Shear Walls (No ERD)	0.30	0.367	0.544	0.670	0.86
	Shear Walls (M-ERD)	0.14	0.210	0.384	0.510	0.70
	Shear Walls (H-ERD)	−0.02	0.047	0.224	0.350	0.54
Steel	Steel Structures	−0.02	0.170	0.324	0.480	0.70
Wood	Wooden Structures	0.14	0.207	0.447	0.640	0.86

*ERD: Earthquake Resistance Design; M: Moderate; H: High.

Table 7
Mean damage-grade equations used by researchers.

Mean damage grade equation	References
$\mu_D = 2.5 \times \left[1 + \tanh\left(\frac{I + 6.25 \times V - 13.1}{Q}\right)\right] \times f(V, I)$	[32,33,34,36,39,40,41,42]
$\mu_D = 2.5 + 3 \times \tanh\left(\frac{I + 6.25 \times V - 12.7}{Q}\right) \times f(V, I)$	[38,43]

the correlation between the economic damage index (d_e)and the mean damage grade (μ_D) a simplified expression was developed (Eq. (15)):

$$\mu_D = 4 \times d_e^{0.45} \tag{15}$$

Table 8 displays some of the relationships between the economic damage index (d_e) and the mean damage grade(μ_D). The index of economic damage ranges from 0 (no damage) and 1 (collapse). As noted, for different methodologies used, the values of economic indicators related to specific damage grades are different.

After defining the conversion of the peak ground acceleration (PGA) or (y) into the intensity scale of EMS-98 given in Eq. (9), and the transformation of the economic damage index (de), into the mean damage grade (μ_D), then the correlation between the vulnerability index, (Iv) in the GNDT II approach and the vulnerability index, (V) that were used in the Macro-seismic approach was possible to be derived based on an expression shown by the equations in Table 9. This correlation approach is limited in use since it is considered only for masonry structures.

According to this correlation, the vulnerability index (Iv) of the GNDT II can be calibrated into the vulnerability index (V) of the European Macro-seismic approach, allowing the computation of the mean damage grade (μ_D) by equations used in Table 10. In addition to what is mentioned, the vulnerability index can be well-defined as a function of the vulnerability classes according to the EMS-98 scale, for example, reinforced concrete buildings are generally included between vulnerability class C, D, E and sometimes F, while masonry buildings are classified in the most vulnerability classes between A and C. The correlation of the vulnerability index in both approaches with respect to vulnerability classes are tabulated in Table 10. Additionally, Giovinazzi and Lagomarsino [33] defined the Vulnerability index (V) for each Macro-seismic vulnerability class as illustrated in Table 11.

Furthermore, most of the cities present a heterogeneous mix between masonry and reinforced concrete elements. From this point of view, a correlation is derived between the vulnerability index of the GNDT II approach and the Macro-seismic approach for RC-buildings. The vulnerability index as a method was applied to 91 existing RC-buildings in different countries in the world such as Japan, China, Italy, Peru, Spain, Turkey, the United States, Haiti, New Zealand, Indonesia, Mexico and Algeria [49]. After that, a new vulnerability function named, mean damage gradesμ_D was adjusted for RC-building typology, that combined the macro-seismic intensity (I_{EMS-98}) and vulnerability index, V as shown in the following equation:

Table 9
Correlations between V and Iv indices used by researchers.

V and Iv correlation expressions	References
$V = 0.56 + 0.0064 \times Iv$	[39,43]
$V = 0.58 + 0.0064 \times Iv$	[42]
$V = 0.592 + 0.0057 \times Iv$	[38,41,48]
$V = 0.46 + 0.012 \times Iv$	[176,179]
$V = 0.46 + 0.0056 \times Iv$	[177]

Table 10
Masonry buildings correlation between two methodologies [43].

GNDT II Method (Iv)	50	25	0
Macro-seismic Method (V)	0.88	0.72	0.56
EMS-98 Vulnerability Class	Class A	Class B	Class C

$$\mu_D = 2.838 \times \left[1 + \tanh\left(\frac{I + 10.79 \times V - 11.6}{Q}\right)\right] \tag{16}$$

The proposed correlation is a quadratic correlation unlike the linear correlation used for masonry buildings as reported by Basaglia et al. [48] in Eq. (17).

$$V = 0.8568 - 0.0083 \times Iv - 0.000039 \times Iv^2 \tag{17}$$

2.3. Seismic assessment of case studies using vulnerability index approaches

Several researchers have applied the above-mentioned methodologies (GNDT and European macro-seismic) in many urban cities to obtain the seismic risk assessment. The following studies have considered adopting the mentioned methods with some modifications considered for each case study. In the following section, different case studies are mentioned to be presented carefully such as Barcelona (Spain), Portugal, Italy, Algeria, Morocco.

2.3.1. Spain, vulnerability assessment using the RISK-UE method

Barcelona was considered as a case study by Lantada et al. [34] to use the RISK-UE framework to propose the seismic risk assessment of the city. An advanced GIS technique was used to draw up the information on a seismic map that identifies all the indices needed in the study. The vulnerability index (V) was applied based on the available data through the Municipal Informatics Institute of Barcelona Government. According to the Handbook of Work package 1 of RISK-UE Project which classified the building typologies, for example in Barcelona, the predominant typologies were masonry and RC- buildings [50]. As a result, the vulnerability index of masonry buildings showed values ranging from 0.70 to almost 1.00, while reinforced concrete buildings were less vulnerable ranging from 0.40 to 0.85. These values have been confirmed by Aguilar-Melendaz et al. [178] that the buildings in Barcelona are highly vulnerable with a mean vulnerability index 0.79.

Table 8
Correlation in distinct methodologies between the mean damage grade level and the economic damage index [43].

Damage Grade	0	1	2	3	4	5
Level of damage	No damage	Slight	Moderate	Severe	Very Severe	Destruction
Economic damage index, d_e						
(ATC, 1985) [44]	0.00	0.050	0.200	0.550	0.90	1.00
(Bramerini et al., 1995) [45]	0.00	0.010	0.100	0.350	0.75	1.00
(Hazus, 1999) [46]	0.00	0.020	0.100	0.500	1.00	1.00
(Dolce et al., 2000) [47]	0.00	0.035	0.145	0.305	0.80	1.00

Table 11
Vulnerability values for each class in RISK_UE approach [32].

EMS-98 Vulnerability Class	Class A	Class B	Class C	Class D	Class E	Class F
Macro-seismic Method (V)	0.88	0.72	0.56	0.40	0.24	0.08

2.3.2. Portugal, vulnerability assessment using GNDT method

In Portugal, the seismic assessment of old masonry buildings is necessary to be studied not only because of their historical heritage but due to their typology characteristic as an ordinary resisting system. Ferreira et al. [38] assessed 192 masonry buildings in the old city Horta, Azores, using the vulnerability index formulation, after the damage data collected from the 1998 Azores earthquake that hit Portugal reaching VII intensity scale. This was proposed by applying the GNDT methodology with some modifications that lead the vulnerability index to be calculated based on 14 parameters instead of 11 parameters [43]. As a result, the first 50 buildings out of 192 were assessed in detail with a mean seismic vulnerability index, Iv_{mean}, of (33.83), while the remaining 142 buildings were assessed with some missing data or incomplete information that resulted in a mean vulnerability index, Iv_{mean}, of (35.92). Furthermore, 8% of the evaluated buildings had an index of above 45, and 4% below 20. While the lowest and highest values that attained from the assessment were (13.65) and (80.38), respectively, and were classified in the vulnerability classes A and B. A similar approach was applied but this time to Seixal old city center that has the same structural typologies of masonry buildings. The seismic vulnerability assessment was assigned to (504 units) that were divided into 3 groups of study [51]. The 1st group consisted of (99 units) with all the possible information was available, while the 2nd group consisted of (197 units) that were still under inspection and the detailed information was not complete. The 3rd group that consisted of (208 units) was not considered in their study. According to the application of the vulnerability index to the first group (99 buildings out of 504) the average vulnerability index value, Iv was calculated as (34.16), and for the second group (197 buildings out of 504) the vulnerability index, Iv has been (32.81). Based on the EMS-98 scale, the buildings were classified between vulnerability classes A and B.

Similarly, it was applied to another old city namely; Faro. This case study was investigated by Maio et al. and Vicente et al. [37,41]. To achieve the seismic vulnerability assessment, a vulnerability index method was used to assess the fragility of 354 buildings. The assessment was started in two stages. In the 1st stage, evaluation of the buildings with complete and detailed information (53 buildings out of 354). In the 2nd stage, the evaluation of the remaining buildings with little information was conducted. After that, the results were obtained using the geographical information system (GIS) tool. As a result, the first 53 buildings out of 354 (1st stage) were assessed in detail with a mean vulnerability index, Iv_{mean}, of (36.15), while for the remaining buildings, their mean vulnerability index, Iv_{mean}, decreased to (34.12).

Still, in Portugal, but with another case study, Coimbra city known as a historic city in Portugal which is dominated by old limestone masonry buildings were assessed by Vicente et al. by using the vulnerability index approach based on the GNDT method but with three more parameters as modifications. Over 679 limestone masonry buildings were assessed by considering two groups of studies. The 1st group consisted of 410 buildings out of 679 buildings, where the detailed information was available. The 2nd group consisted of 269 buildings with a lack of information during the assessment. The 410 assessed buildings resulted in mean vulnerability index Iv_{mean} of (38.13), while in the 2nd group, it shifted up to (38.38). The highest and lowest Iv values obtained for the whole studied buildings were (60.58) and (12.12), respectively.

2.3.3. Italy, vulnerability assessment using GNDT method

In Italy, Lampedusa Island in southern Italy was studied by Cavaleri et al. [52] to develop the vulnerability index based on the GNDT method. The seismic vulnerability assessment investigated 288 buildings, which consisted of 264 masonry buildings and 24 RC-buildings. The data collected by field surveys could be allowed to achieve the judgment on the general vulnerability conditions of the island. The vulnerability output was generated using the GIS tool, and the result of the mean vulnerability index was relatively low, meaning it was in good seismic performance, compared to the normal value which is equal to (25.6).

2.3.4. Morocco, vulnerability assessment using the RISK-UE method

In north Morocco, Al Hoceima city is demonstrated as one of the highest seismic active zones in Morocco. Cherif et al. [53] estimated the vulnerability of the current buildings through the European approach. Particularly, the majority of the constructed buildings were reinforced concrete buildings as stated by Lungu et al. [54]. According to the result, the studied buildings had a mean vulnerability index of (0.49), with minimum and maximum values of (0.2) and (0.9), respectively. Similarly as estimated by Cherif et al. in Imzouren city northern Morocco [55].

2.3.5. Algeria, vulnerability assessment using GNDT and RISK-UE method

In northeast Algeria, Annaba city is classified as a moderate seismic hazard as stated by Boughacha et al. [56], Peláez et al. [57], Kherroubi et al. [58], and Mourabit et al. [59]. In this case study, Athmani et al. [36] applied two seismic vulnerability index methods to analyze the vulnerability of the masonry buildings adapted in the Annaba city. The data of the masonry buildings were gathered depending on a team of experts in the CTC (Control Technical Construction). The CTC data for Annaba city classified the buildings into four classes namely; Green; Yellow; Orange; and Red. In the 1st class consisted of buildings that do not require any rehabilitation interventions. The 2nd class consisted of buildings that required slight repair or strengthening interventions. The 3rd class consisted of buildings that required serious repairs. The 4th class consisted of buildings that required to be demolished or reconstructed. As a result, the application of the two approaches was integrated into the GIS Map. The average vulnerability index value was equal to (0.91), with the highest and lowest values of (1.02) and (0.55), respectively, for the whole selected studied buildings in Annaba city. With regards to that, for the GNDT approach, the vulnerability index (Iv) value was more than (45) with vulnerability class A.

2.3.6. Seismic vulnerability index for the steel structures

The structural and mechanical properties of steel structures are needed to identify the seismic vulnerability assessment using the Vulnerability Index method. Mahmoud [60] developed a vulnerability index method for steel structures by studying two case studies. These two studies were classified into three classes namely Green, Orange, and Red. The 1st class represents that the construction may not require any repair. The 2nd class represents that the construction required moderated repair/retrofit work, while the 3rd class classified the building as very vulnerable and should be demolished, as illustrated in Table 12.

Table 12
Vulnerability Index Classes.

Vulnerability Class	Green	Orange	Red
Vulnerability Index (VI)	[0.36–0.54]	[0.54–0.85]	[0.85–1.00]

2.3.7. Case study: data summary

The vulnerability assessments using a vulnerability index of the case studies using the RISK-UE method are tabulated in Table 13, and the previously mentioned study areas are summarized in Table 14.

3. Analytical assessment approach

The analytical procedures for determining the seismic physical vulnerability of structures may also be named as the theoretical approaches, since, in contrast to the empirical approach (vulnerability index + expert judgment, RVS), which are based on observations, they rather focus on simulating the strong ground motions. There are several analytical methods to accurately assess the behavior and performance of building structures during earthquake movements. The analytical approaches involve linear static, linear dynamic, nonlinear static, and nonlinear dynamics analyses. There has recently been increasing awareness and interest in designing structures exposed to an earthquake or seismic action based on the seismic regulations or in the performance-based design. To precisely assess the seismic demands of structures, the nonlinear analysis is the method that is usually required to be used. Generally, it can be categorized into two groups: Non-Linear Time History Analysis (NLTHA), and Non-Linear Static or Pushover Analysis (NLSA/POA).

3.1. Non-Linear static analysis (NLSA) - Pushover analysis (POA)

In the earthquake engineering field, the non-linear static analysis method has become very popular due to its simplicity. It becomes an active engineering tool for estimating the structural safety against earthquake struck-induced collapse. This method was initially presented in FEMA 273 [62] where the "Coefficient Method" has been used to determine the target displacement and then it was updated in FEMA 356 [63]. The non-linear static analysis refers to the pushover analysis that will result in a well-known curve identified as "Capacity Curve". The ultimate goal of this approach is to obtain the structure's dynamic properties such as stiffness, strength, and ductility under seismic loading.

In non-linear static analysis or simply the POA procedure, the constructed model of the structure will consider explicitly the non-linear force and displacement behavior of its structural elements. After that, a relationship would be developed between base shear and displacement (V vs. Δ) via exposing the structure to lateral forces monotonically increasing until the displacement of the model exceeded or reached the allowable displacement that described a predefined structural damage. As a definition, the allowable displacement is known as the target displacement. A global failure could happen when the slope of the curve becomes negative. From this method, the in-elastic response behavior can be determined for an equivalent single degree of freedom (SDOF). This implies the need to transform multi-degree of freedom (MDOF) into a single degree of freedom that limits the applicability of this approach. However, this transformation would be exact only if the structure is vibrated in a single mode with constant deforming shape over time.

It was found that the procedure has some rigorous lacks in its theoretical foundation. The procedure as mentioned was based on two assumptions, firstly the structural responses were conquered by the fundamental vibration mode, secondly, the displacement vector remained constant [64]. These could be incorrect and not always fulfilled, and the structures nonlinear response could not be built on the first mode vibration and the constant lateral forces distributed (Triangular or Rectangular) over the height of the structure [65]. Meanwhile, it neglected the duration and cyclic influences as well as the dynamic features of the structure. Some researchers found that the procedure did not provide a precise result compared to non-linear time history analysis or either experimentally in evaluating building seismic behavior (Gupta and Kunnath [66], Chopra and Chintanapakdee [67], Goel and Chopra [68] Maison and Bonowitz [69]). This procedure may be doubtful to be used unless it could predict the capability of the structure

Table 13
Vulnerability Index in Europe countries using the RISK-UE Method [28].

Country	Romania	Macedonia	Italy	France	Bulgaria
City	**Bucharest**	**Bitola**	**Catania**	**Nice**	**Sofia**
Nb. of buildings in the city	108,834	13,657	41,800	37,000	3865
Nb. of buildings whereby analyses are carried out	4068	13,657	41,800	3328	3865
Minimum Vi, Masonry	0.202	0.623	0.509	0.301	0.3
Maximum Vi, Masonry	0.365	0.825	0.804	1.02	1.02
Minimum Vi, Reinforced Concrete (RC)	0.068	0.542	0.414	0.247	−0.02
Maximum Vi, Reinforced Concrete (RC)	0.257	0.623	0.759	0.782	1.02

Table 14
Vulnerability Index case studies using GNDT II and RISK-UE methods.

Country	City	Typology	Lower VI	Mean VI	Upper VI	Empirical Approach	References
Portugal	Horta	URM	13.65	34.87	80.38	GNDT II	[38]
	Seixal	URM	15.00	33.48	63.00	GNDT II	[51]
	Faro	URM	20.00	35.14	53.00	GNDT II	[41,42]
	Coimbra	URM	12.12	38.26	65.58	GNDT II	[43]
Spain	Barcelona	RC	0.4	0.65	0.85	RISK-UE	[34]
		URM	0.7	0.87	1.00		
Italy	Lempedusa	RC & URM	–	25.6	–	GNDT II	[52,61]
Morocco	Al-Hoceima	RC	0.2	0.9	0.49	RISK-UE	[53,55]
Algeria	Annaba	URM	0.55	1.02	0.91	GNDT II & RISK-UE	[36]

*-: Not mentioned in the study, or a few details of information and data to study the vulnerability index in the field.
*RC: Reinforced Concrete.

and estimate the safety limit states against the total failure. Nevertheless, this method has been used in a sequence of studies in assessing the structural capacity (Zacharenaki et al. [70], Fragiadakis and Vamvatsikos [71], Shafei et al. [72], Fiore et al. [73], Zameeruddin and Sangle [74]). In recognition of these doubtful deficiencies, the non-linear static analysis was modified to achieve better seismic demand estimation, where too many things have been done to take into consideration such as the contribution of higher modes, torsional effect, redistribution of inertia forces, and irregular structures. The modification procedures have been as follows: Modal Pushover Analysis (MPA) procedure and Modified Modal Pushover Analysis (MMPA), Adaptive Modal Pushover Analysis (AMPA) procedure, Consecutive Modal Pushover (CMP) procedure and Modified Consecutive Modal Pushover (MCMP) procedure, Extended N2 procedure and the Envelope-based Pushover procedure, and Improved Modal Pushover Analysis (IMPA) procedure. Recently, Liu and Kuang [75] proposed a procedure to evaluate the seismic performance and demand for tall buildings, namely Spectrum-Based Pushover Analysis (SPA). The methodology was applied on two steel structures with a special moment-resisting frame detailing with 9 and 20 stories, and the results were compared with (NLTHA) analysis that showed a good agreement. In addition, Rahmani et al. [76] proposed a non-linear static procedure called improved upper bound (IUB) pushover analysis. The purpose of this method when analyzing irregular tall buildings is to be precise and accurate that could correlate and get closer to (NLTHA). Table 15 shows the prior modified methods of the non-linear static analysis and its purposes.

Table 15
Modified pushover methods and their purposes.

Modified Pushover Method	Purpose	Authors	References
Modal Pushover Analysis (MPA)	To provide the ability in the contribution of all vibration modes that have a significant influence on the seismic response especially in tall buildings.	Chopra and Goel	[77]
Modified Modal Pushover Analysis (MMPA)	The MPA has been extended to assess the seismic demand capacity considering the higher modes in elevation.	Chopra and Chintanapakdee	[67]
Adaptive Modal Pushover Analysis (AMPA)	To redistribute the inertia forces associated with the effects of changing the dynamic features throughout the in-elastic response, as well as considering the influence of higher modes.	[Gupta and Kunnath; Kalkan and Kunnath; Antoniou and Pinho]	[66,78,79]
Consecutive Modal Pushover Analysis (CMP)	To evaluate the seismic demand of tall buildings, and the seismic performance of different modes and the force vector for each mode applied consecutively to the structure.	Poursha et al.	[80]
Modified Consecutive Modal Pushover Analysis (MCMP)	The CMP has been extended to estimate the seismic demand based on the consequences of higher modes and torsion.	[Poursha et al.; Khoshnoudian and Kiani]	[81,82]
Extended N2 method	To consider both torsional and higher mode effects.	[Kreslin and Fajfar; Brozovič and Dolšek; Fajfar]	[83,84,85]
Improved Model Pushover Analysis (IMPA)	(IMPA) is a multi-mode approach that has the capability to re-define the lateral loads, and to realize the in-elastic deformed shape rather than the elastic deformed shape.	[Poursha and Samarin; Belejo and Bento]	[86,87]
Improved upper bound (IUB) pushover analysis	The IUB proposed an adjustment of lateral load pattern applied to high-rise buildings, and improved accuracy in evaluating the seismic response of high-rise buildings.	Rahmani et al.	[76]
Spectrum-Based Pushover Analysis (SPA).	The SPA procedure has the ability to estimate the seismic demand of high-rise buildings quickly and accurately, which is nearly similar to NTHA accuracy.	Liu and Kuang	[75]

3.2. Non-Linear time history analysis (NLTHA) - incremental dynamic analysis (IDA)

The NLTHA is the most exact and precise method to assess the seismic performance of a structure/infrastructure. Recently, the computational methods were in rapid development, and the incremental dynamic analysis (IDA) as an improved and extended version of NLTHA methodology has become a powerful tool in evaluating the dynamic behavior of the structures subjected to earthquake motions. It was proposed as early as in 1977 by Bertero [88] and after that, it was studied extensively by several researchers and investigators (Bazzurro and Cornell [89], Bazzurro et al. [90], Vamvatsikos and Cornell [91], Yun et al. [92], Lin and Baker [93], Jalayer et al. [94], and Miano et al. [95]).

Also, it was approved by FEMA 2000 as a technique to investigate the global collapse capacity. Incremental dynamic analyses have lately played a significant role in studying the general behavior of the structures, starting from the elastic response stage through yielding and non-linear response stages, until reaching the instability of the structure. Moreover, IDA gave a noticeable vision about the performance of a structure under seismic actions. Thus, a set of ground motion records based on (NLTHA) is usually needed to develop an incremental dynamic analysis (IDA). In which the ground motion intensity was selected for investigating the structural performance. This could be done by applying a successive incrementally increase of the seismic intensity until the structure reaches the global collapse capacity. The IDA result can be depicted by plotting the ground motion intensity (IM) vs. a structural response parameter (EDP).

In the last decades, many researchers have been using incremental dynamic analysis as the main tool in their research, the studies are summarized in Table 16.

The main advantages of this method are the capacity to model wide diversity of non-linear material behavior, irregularity in structures with geometric non-linearity, pounding buildings behavior, and higher mode effects in tall buildings that can be done precisely only with the non-linear dynamic procedure. However, this type of analysis also has disadvantages such as; it needs a complex platform to create the analytical model, consuming time to accomplish the analysis, lack of supercomputers readily to do the analysis, and a large number of ground motions are necessary to perform the analysis as mentioned by (Roca [111], Shome [112], Krawinkler et al. [113], Bakhshi and Asadi [114]). Positive and negative feedbacks for this method were also reported by Silva et al.

Table 16
Summary of previous studies on NLTHA.

Authors	Study Highlight	Intensity Measure (IM)	Engineering Demand Parameter (EDP)	Software	References
Vamvatsikos	Developed the IDA curves to investigate the structural behavior response of a 20-story steel space frame system under a set of earthquake records.	Sa (T1,5%), g	Inter-story drift ratio ϴmax	OpenSees	[96]
Kirçil and Polat	Developed the IDA curves for 3, 5, and 7 stories reinforced concrete buildings in Turkey (Istanbul).	Sa (T1,5%), g	Inter-story drift ratio	IDARC	[97]
Asgarian et al.	Investigated the seismic performance of three types of beam-column joint connection for a steel moment-resisting frame system namely; Special, Intermediate, Ordinary.	Sa (T1,5%), g	Inter-story drift ratio ϴmax	OpenSees	[98]
Farsangi et al.	The seismic reliability of a steel frame has been evaluated in high seismicity regions.	PGA	Inter-story drift ratio ϴmax	OpenSees	[99]
Fanaie and Ezzatshoar	Studied the dynamic properties of the concentric steel bracing system called Gate bracing system.	Sa (T1,5%), g	Inter-story drift ratio ϴmax	OpenSees	[100]
Nazri and Saruddin, Saruddin and Nazri	Employed the IDA curves to assess the seismic behavior in Malaysia for low and mid-rise buildings.	PGA	Drift (%)	SAP2000	[101,102]
Farsangi and Tasnimi	The influence of combined horizontal-vertical ground excitations has been studied on non-ductile RC frame structures.	Sa (T1,5%), g	Inter-story drift ratio ϴmax	OpenSees	[103]
Farsangi et al.	The collapse margins of an array of RC structures were evaluated based on a proposed vector IM under the influence of multi-component earthquake excitations.	Vector IM	Inter-story drift ratio ϴmax	OpenSees	[104]
Gonzalez-Drigo et al.	Analyzed the vulnerability of the URM buildings in Barcelona using IDA curves.	PGA	Maximum Displacement	TreMuri	[105]
Fathieh and Mercan	Suggested to study the seismic demand and the capacity of 4-story MSB using the Incremental dynamic analysis method.	Sa (T1,5%), g	Maximum Inter-story drift ratio (%)	OpenSees	[106]
Sobhan et al.	Investigated the dynamic buckling behavior of a steel cylindrical tank using (NSPO) and then compared with (NLTHA) to assess the accuracy of the result.	PGA	Maximum Radial Displacement	ABAQUS	[107]
Moazam et al.	Suggested to assess the seismic demand capacity of two old concrete arch bridges in Iran. The analyses have been investigated using non-linear dynamic analysis by simulating 22 far-field earthquake records.	PGA	Maximum Displacement	ANSYS	[108]
Azizan et al.	Suggested to study the seismic performance of Koyna dam which is classified as a concrete gravity dam. The assessment was performed by generating the incremental dynamic analysis under single and repeated earthquake excitations.	PGA	Maximum Crest Displacement	ABAQUS	[109]
Kildashti et al.	Introduced a liquid storage tank to investigate the influence of the base connection under seismic movements (22 records).	PGA	Maximum Radial Displacement	ABAQUS	[110]

[115]. Muntasir Billah and Shahria Alam [116] stated that the analysis required many earthquake records that made the computational analysis extensive. Nevertheless, Colapietro et al., Farsangi et al., Farsangi and Tasnimi considered this method as the most conservative method compared to non-linear static analysis, and more accurate in predicting the structural responses [117,99,103].

4. Analytical vulnerability/fragility curve

As stated before, the vulnerability curves were generally derived by using the observational damage data of previous events, but recently the computational analyses were much more reachable to develop this type of curve. The fragility curves or the vulnerability curves were analytically used to evaluate the risk of the earthquake effect on the building structures. It was considered as a valuable tool to predict damage possibilities that may influence the structures. Also, it can be used as an indicator in the rehabilitation and retrofitting planning. The earthquake and its ground motion have a huge catastrophic effect on the structural behavior, for that reason, implementing the fragility analysis besides the non-linear analysis is the most beneficial tool to estimate the structural responses and the financial losses. The vulnerability curves are mostly developed for the vulnerability of residential buildings, that have been constructed as reinforced concrete (RC), steel and masonry structures [118–124]. However, the prefabricated reinforced concrete structures have been well considered in recent years [125–131]. Furthermore, the infrastructures were also investigated by some researchers (Shinozuka et al. [132], Siqueira et al.[133], Alessandri et al.[134], Long et al. [135], Segura et al. [136], D'Amico and Buratti [137]). Hence, these curves will help as an indicator before and after earthquake events, as well as in developing upcoming code provisions.

The vulnerability curves were developed with the aid of non-linear analysis such as IDA and POA. Based on previous researches, most of the studies used the incremental dynamic analysis (IDA) as the first stage in developing the fragility curves more than that of the pushover analysis (POA). For example, Vona [138] developed the fragility curves to examine the seismic structural response of the moment resisting concrete frame (MRCF) using two distinct analytical methods, namely, (NSA) and (NDA). With regards to this, it shows that the NDA was the greatest method to consider. Anvarsamarin et al. [139] estimated the collapse performance of three structural models of 6, 12, and 18 as an RC-MRF with different story heights. The estimation was developed by using the fragility curves and by considering the soil-structure interaction as a seismic uncertainty parameter. Tajammolian et al. [140] analyzed the seismic efficiency of asymmetric steel structures isolated with Triple Concave Friction Pendulum (TCFP) as a bearing seismic element. The fragility curves have been developed after performing the IDA analysis under the effect of 45 sets of artificial seismic records. Then, the damage states of HAZUS-2003 were used to consider the damage probabilities.

Nazari and Saatcioglu [141] investigated the seismic vulnerability of RC-shear wall buildings in Vancouver by involving the non-linear time history analysis (NLTHA) using 20 artificial seismic records, then the fragility curves were provided to detect the damage levels imposed to buildings according to ASCE41[142]. Karapetrou et al. [143] discussed the seismic performance of reinforced concrete buildings by considering the aging effect.

Through modeling, the corrosion time rate using OpenSees software was measured; the non-corroded time was defined as T = 0 years, while the corroded time was taken at (T = 25, T = 50, and T = 75 years). After applying the 15 real ground motion records implementing the IDA approach, the fragility curves were derived according to HAZUS prescriptions. Banazadeh and Ghanbari [144] studied the seismic collapse performance of three steel moment-resisting frames with 6, 8, and 12 stories designed as special steel moment frames with and without non-linear viscous dampers in accordance with the ASCE code provisions. The fragility curves were performed to examine the probability of collapse under the effect of ground motion far-field records. Table 17 presents other researches and studies in deriving analytical fragility curves. According to Billah and Alam, the fragility general equation was expressed in Equation (18).

$$Fragility = P[LS|IM = y] \tag{18}$$

where LS refers to the damage limit state, IM is the ground motion intensity measure, and Y is the ground motion intensity.

Even though the fragility curves were generated by several equations, these types of curves were known as standard normal cumulative distribution function, that have distinct variations and parameters for each study such as earthquake motion intensity and the measure of damage [169–172]. However, the simplest one is the one that was used by Kircil and Polat [97], and Ibrahim and El-Shami [173] as shown in Eq. (19):

$$P(x) = \phi\left(\frac{\ln X - \lambda}{\varsigma}\right) \tag{19}$$

where Φ is the cumulative distribution function, λ is the mean value, and ς is the standard deviation.

One of the guideline drawbacks of the analytical vulnerability curves is that the procedure is greatly computational and time-consuming; therefore, the fragility curves cannot be easily developed because of the large number of uncertainties to be considered in the modeling procedure. Nevertheless, the analytical fragility curves have been utilized to support and probably replace the empirical vulnerability curves due to the lack of data related to post-earthquake events which caused the damages, as well as the observational damages. Table 18 shows the advantages and disadvantages of empirical and analytical methods.

Table 18
Advantages and disadvantages of the empirical and analytical methods in developing vulnerability curves.

Method	Advantages	Disadvantages
Empirical	Observational damage during the event which shows realistic vulnerabilities.	Missing data or lack of data, Not clear vision to investigate the damages, not accurate, and mainly depend on expert decisions with different opinions.
Analytical	The most accurate method, all type of uncertainties can be considered	Time-consuming, very sensitive to modeling and analysis approach, and computational inefficient

Eventually, to develop vulnerability curves, four distinct methods can be utilized, namely: (1); Expert and judgment approach, (2); Empirical approach, (3); Analytical approach, (4); Hybrid approach. We can differentiate between the four general curve types [174].

1. Empirical curves based on the observed seismic damage information and data.
2. Judgment curves based on skilled judgment and opinion.
3. Analytical curve based on simulated damage data.
4. Hybrid curve based on a grouping of the previous methods.

Fig. 3 below describes the flow-chart methodologies to develop fragility curves.

5. Needs and challenges

From the literature reviews, it can be derived that the seismic evaluation approaches have been used to specify and estimate the structure's capacity to withstand an earthquake (pre or post-

Table 17
Analytical vulnerability curves developed by researchers.

Structural System	Type of Analysis	Engineering Demand Parameter (EDP)	Intensity Measure (IM)	Reference
MRCF	NLS	Top displacement	Sd(T)	[119]
URM	NLD + NLS	Interstory drift (ISD)	PGA	[145]
URM	NLS	Top displacement (TD)	Sd(T)	[146]
URM	NLS	Base + Flexural Strength	PGA	[147]
MRCF	NLS	Interstory drift (ISD)	PGA	[148]
URM	NLS	Top displacement (TD)	Sd(T)	[149]
MRCF	NLS	Top Displacement (TD)	Sd(T) and PGA	[150]
RC-dual frame-wall system	NLD	Interstory drift (ISD)	PGA, PGV, and PGD	[151]
MRCF + Dual-frame-wall system	NLS	Chord rotation, Shear force	PGA	[152]
MRCF + Dual-frame-wall system	NLS	Interstory drift (ISD)	PGV	[153]
URM	LS	Extent of damage	Sa(T)	[154]
URM	NLS	Interstory drift (ISD)	PGA	[155]
MRCF + Dual-frame-wall system	NLS	Chord rotation, Shear force	PGA	[156]
RC + URM	NLD + NLS	Interstory drift (ISD)	Sa (T1,5%), g	[157]
URM	NLS	Top Displacement (TD)	Sa (T1,5%), g	[158]
RC infilled frames	NLS	Top Displacement (TD)	PGA	[159]
RC (Precast)	NLD + NLS	Maximum Top Drift	Sa(T)	[128]
MRCF	NLD	Interstory drift (ISD)	PGA, PGV, PGD, Sa(T), Sd(T), and Sv(T)	[160]
Monument Masonry wall	NLD	Crack propagation	Sa (T1,5%), g	[161]
MRCF	NLD + NLS	Top Displacement (TD)	Sa (T1,5%), g	[162]
RC-Shear Walls	NLD	Interstory drift (ISD)	Sa (T1,5%), g	[163]
Steel Bridge	NLD + NLS	Base Shear, Lateral Displacement	PGA	[164]
Confined Masonry Walls	NLS	Top Displacement	Sa (T1,5%), g	[165]
Gravity Dam	NLD	fundamental Period	Sa(T), PGV	[136]
RC Shear Walls	NLS + NLD	Moment-Curvature	PGV	[166]
Masonry Walls	NLD	Top Displacement	PGA, Sa(T1)	[167]
MRCF	NLD	Interstory drift (ISD)	PGA	[168]

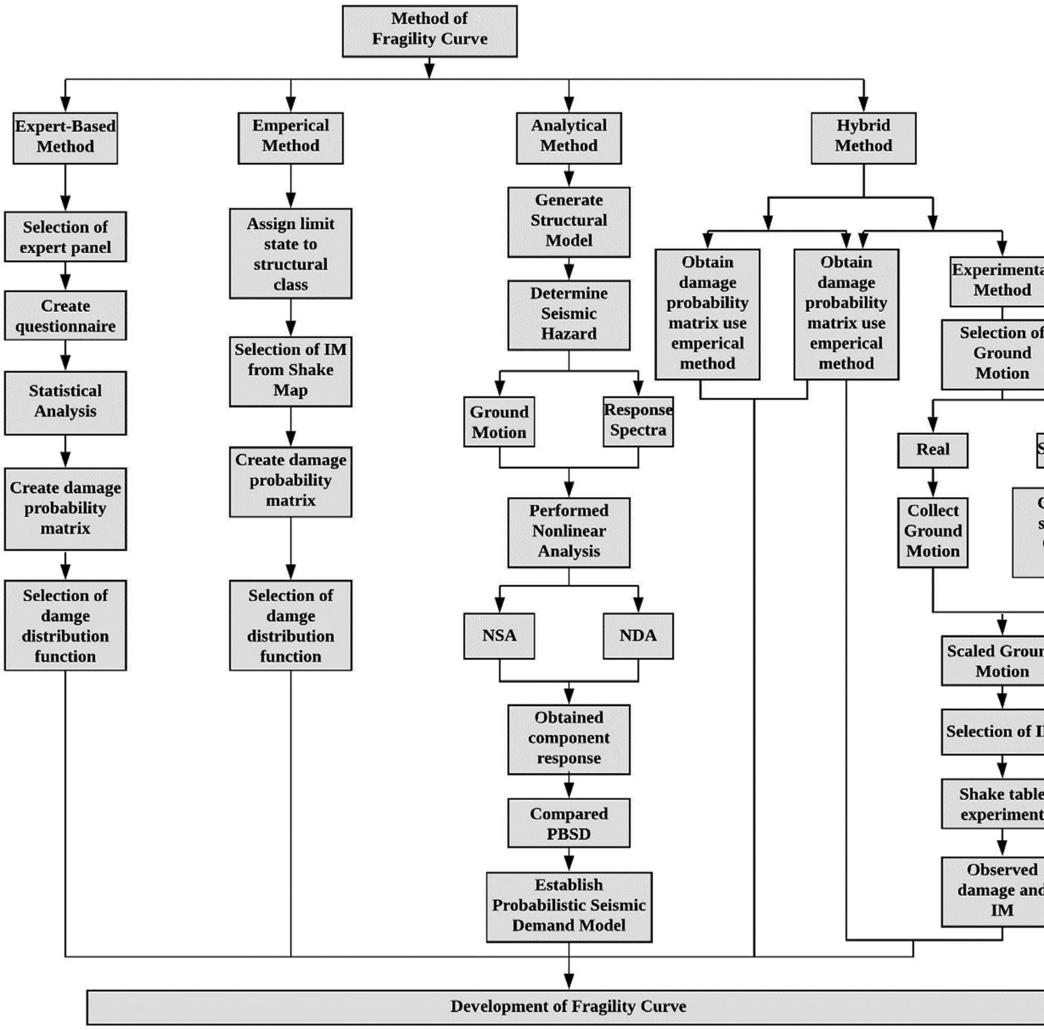

Fig. 3. Methodologies and steps to develop fragility curves [175].

event). However, the task could be very complicated. Accordingly, the empirical methods such as Rapid Visual Screening (RVS), and the vulnerability index approach seem to be unreliable tools because, as discussed earlier, they were based on the observed damage data which may be limited or inaccurate. On the other hand, they do not cover all building typologies, intensities and vulnerability parameters, and the outcome is very dependent on expert judgment. These parameters may significantly affect the collapse of the structure, moreover, they have inconsistency and variations in weighing the parameter to classify the vulnerability of a particular structure as mentioned in the previous case studies.

Analytically, there are still a few extraordinary issues to be addressed with the analytical strategies, such as the potential and the ability of the quantitative models to precisely foresee the behavioral response of the real and actual structure, the precision in converting numerical computational models into real structural damage and vice versa. The tendency of considering human mistakes in the seismic design and executing buildings are the main reasons of disastrous collapses, which need to convert the observational damages and the uncertainties errors during construction into computational modeling to classify the structural vulnerability stages. Therefore, it seems that accuracy in predicting the vulnerability level of a structure could be possible if the prediction is obtained by using finite element modeling and by applying the non-linear analysis tools. Further research challenges include the correlation or transformation of field damages by applying an

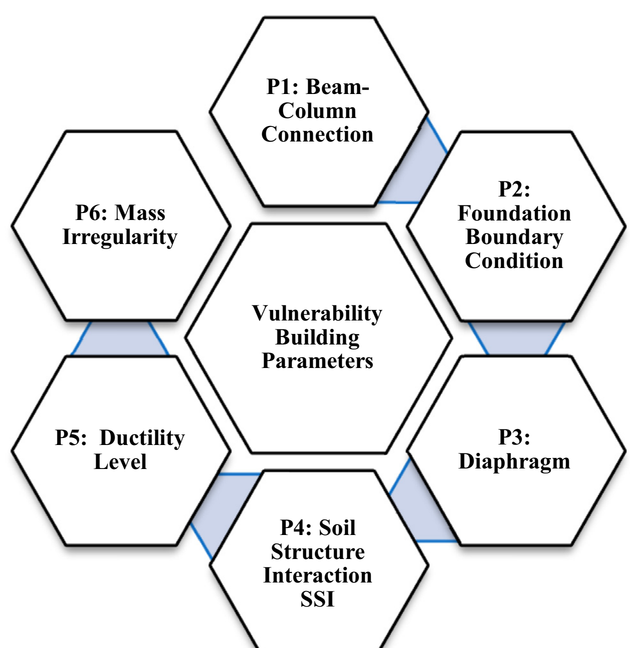

Fig. 4. Description the building vulnerability parameters as physical measurements for structural responses.

Table 19
Parameters used in assessing the building structure's vulnerability.

Parameters Influencing the Building Response		Methods for assessing vulnerability					
		Empirical Assessmesnt		Expert-Judgment Assessment		Analytical Assessment	
		Required	Desirable	Required	Desirable	Required	Desirable
Construction System	Type of load-bearing elements	✓		✓		✓	
	Type of non-load-bearing elements		✓		✓	✓	
Dimension properties	Number of stories					✓	
	Load-bearing elements					✓	
	Non-load bearing elements					✓	
Material properties used in construction	Load-bearing elements					✓	
	Non-load bearing elements						✓
Structural detailing	Load-bearing elements					✓	
	Non-load bearing elements						✓
Age of construction		✓					
Observed damage data from prior earthquakes		✓					

analytical approach. This can be obtained by designing the parameters that have a direct impact to control the structural behavior either by considering the past damage observations or by expecting the damaged spot in the structure. Thus, it can be done by developing an analytical vulnerability index as a measuring criterion for classifying the seismic vulnerability classes by weighting the modeling parameters of a particular building. Many researchers have dealt with this concept, as several parameters influenced the physical vulnerability of the structure. These structural parameters are the main elements that contributed to the response of the structure and the consequences will be in terms of financial and human life losses. Fig. 4 describes the building vulnerability parameters as physical measurements for the structural response, and Table 19 illustrates the vulnerability measurement parameters that each method needs (required or desirable) for the seismic vulnerability assessment.

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References

[1] Glaister S, Pinho R. Development of a simplified deformation-based method for seismic vulnerability assessment. J Earthquake Eng 2003;7:107–40.
[2] Daniell J, Vervaeck A. The CATDAT damaging earthquakes database–2010–year in review. CEDIM Earthquake Loss Estimation Series 2011:2011–101.
[3] Masi A, Vona M. Vulnerability assessment of gravity-load designed RC buildings: Evaluation of seismic capacity through non-linear dynamic analyses. Eng Struct 2012;45:257–69.
[4] Council AT. Procedures for Post-earthquake Safety Evaluation of Buildings Applied technology council; 1989.
[5] Bagio C, Bernardini A, Colozza R, Corazza L, Della Bella M, Di Pasquale G, et al. Field Manual for post-earthquake damage and safety assessment and short-term countermeasures (AeDES). European Commission—Joint Res Centre—Inst Protect Secur Citizen, EUR 2007:22868.
[6] Polese M, Di Ludovico M, Marcolini M, Prota A, Manfredi G. Assessing reparability: simple tools for estimation of costs and performance loss of earthquake damaged reinforced concrete buildings. Earthquake Eng Struct Dyn 2015;44:1539–57.
[7] Kircher CA, Nassar AA, Kustu O, Holmes WT. Development of building damage functions for earthquake loss estimation. Earthquake Spectra 1997;13:663–82.
[8] Perrone D, Aiello MA, Pecce M, Rossi F. Rapid visual screening for seismic evaluation of RC hospital buildings. In: Structures. Elsevier; 2015. p. 57–70.
[9] FEMA. FEMA310, Handbook for the seismic evaluation of buildings. Washington DC: Federal Emergency Management Agency; 2005.
[10] FEMA. FEMA154, Rapid visual screening of buildings for potential seismic hazards: a handbook. Government Printing Office; 2017.
[11] Wallace NM, Miller TH. Seismic screening of public facilities in Oregon's western counties. Pract Period Struct Des Constr 2008;13:189–97.
[12] Holmes WT. Progress of Seismic Rehabilitation of Buildings in the US. Improving the Seismic Performance of Existing Buildings and Other Structures; 2010.
[13] Allen D, Rainer J. Guidelines for the seismic evaluation of existing buildings. Can J Civ Eng 1995;22:500–5.
[14] Cheung M, Foo S, Granadino J. Seismic retrofit of existing buildings: innovative alternatives. ICO-MOS International World committee. eISSN 2001:2319–11163.
[15] Saatcioglu M, Shooshtari M, Foo S. Seismic screening of buildings based on the 2010 National Building Code of Canada. Can J Civ Eng 2013;40:483–98.
[16] Otani S. Seismic vulnerability assessment methods for buildings in Japan. Earthquake Eng Eng Seismol 2000;2:47–56.
[17] Albuquerque P. Seismic vulnerability assessment of medium-rise buildings using the Index Method: the case of block# 22 of Santa Maria Hospital; 2008.
[18] Hassan AF, Sozen MA. Seismic vulnerability assessment of low-rise buildings in regions with infrequent earthquakes. ACI Struct J 1997;94:31–9.
[19] Yakut A. Preliminary seismic performance assessment procedure for existing RC buildings. Eng Struct 2004;26:1447–61.
[20] Bal I, Gulay F, Tezcan S. A new approach for the preliminary seismic assessment of RC buildings: P25 Scoring Method. In: Proceedings of 14th WCEE; 2008, p. 12–7.
[21] NZSFE. Assessment and Improvement of the Structural Performance of Buildings in Earthquakes: Prioritisation, Initial Evaluation, Detailed Assessment, Improvement Measures: Recommendations of a NZSEE Study Group on Earthquake Risk Buildings, New Zealand Society for Earthquake Engineering; 2014.
[22] Giovinazzi S, Lagomarsino S. A methodology for the vulnerability analysis of built-up areas. In: Proc. of the international conference on earthquake loss estimation and risk reduction; 2002, p. 3–18.
[23] Benedetti D, Benzoni G, Parisi MA. Seismic vulnerability and risk evaluation for old urban nuclei. Earthquake Eng Struct Dyn 1988;16:183–201.
[24] Terremoti CGND. Rischio sismico di edifici pubblici Parte II Risultati per la regione Emilia-Romagna. Roma: CNR Gruppo Nazionale Difesa Terremoti; 1993.
[25] Benedetti D, Petrini V. On seismic vulnerability of masonry buildings: proposal of an evaluation procedure. L'industria delle Costruzioni 1984;18:66–78.
[26] Faccioli E, Pessina V, Calvi G, Borzi B. A study on damage scenarios for residential buildings in Catania city. J Seismolog 1999;3:327–43.
[27] Guagnti E, Petrini V. lcaso delle vecchie costruzioni: verso una nuova legge danni-intensità. In: Proceedings of the 4th Italian national conference on earthquake engineering. p. 145–53.
[28] Mourous P, Le Brun B. Risk-UE project: an advanced approach to earthquake risk scenarios with application to different European towns. Assessing and managing earthquake risk. Springer; 2008.
[29] Grunthal G. European macroseismic scale 1998. European Seismological Commission (ESC); 1998.
[30] Lagomarsino S, Giovinazzi S. Macroseismic and mechanical models for the vulnerability and damage assessment of current buildings. Bull Earthq Eng 2006;4(4):415–43.
[31] Milutinovic ZV, Trendafiloski GS. Risk-UE An advanced approach to earthquake risk scenarios with applications to different european towns. Contract: EVK4-CT-2000-00014, WP4: Vulnerability of Current Buildings; 2003.
[32] Bernardini A, Giovinazzi S, Lagomarsino S, Parodi S. Vulnerabilità e previsione di danno a scala territoriale secondo una metodologia macrosismica coerente con la scala EMS-98; 2007.
[33] Giovinazzi S, Lagomarsino S. A macroseismic method for the vulnerability assessment of buildings. Proc of 13th World Conference on Earthquake Engineering, 2004.
[34] Lantada N, Irizarry J, Barbat A, Goula X, Roca A, Susagna T, et al. Seismic hazard and risk scenarios for Barcelona, Spain, using the Risk-UE vulnerability index method. Bull Earthq Eng 2010;8:201–29.

[35] Azizi H, Mendes N, Lourenco PB, Hajisadeghi N. A seismic vulnerability index method for masonry schools in the province of Yazd. In: 9th International Masonry Conference; 2014.
[36] Athmani AE, Gouasmia A, Ferreira TM, Vicente R, Khemis A. Seismic vulnerability assessment of historical masonry buildings located in Annaba city (Algeria) using non ad-hoc data survey. Bull Earthq Eng 2015;13:2283–307.
[37] Maio R, Ferreira TM, Vicente R, Estevao J. Seismic vulnerability assessment of historical urban centres: case study of the old city centre of Faro, Portugal. J Risk Res 2016;19:551–80.
[38] Ferreira TM, Maio R, Vicente R. Seismic vulnerability assessment of the old city centre of Horta, Azores: calibration and application of a seismic vulnerability index method. Bull Earthq Eng 2017;15:2879–99.
[39] Athmani A, Ferreira TM, Vicente R. Seismic risk assessment of the historical urban areas of Annaba city, Algeria. Int J Archit Heritage 2018;12:47–62.
[40] Azizi-Bondarabadi H, Mendes N, Lourenco PB, Sadeghi NH. Empirical seismic vulnerability analysis for masonry buildings based on school buildings survey in Iran. Bull Earthq Eng 2016;14:3195–229.
[41] Vicente R, Ferreira T, Maio R. Seismic risk at the urban scale: assessment, mapping and planning. Procedia Econ Finance 2014;18:71–80.
[42] Maio R, Vicente R, Formisani A, Varum H. Seismic vulnerability of building aggregates through hybrid and indirect assessment techniques. Bull Earthq Eng 2015;13:2995–3014.
[43] Vicentte R, Parodi S, Lagomarsino S, Varum H, Silva JM. Seismic vulnerability and risk assessment: case study of the historic city centre of Coimbra, Portugal. Bull Earthq Eng 2011;9:1067–96.
[44] ATC. Earthquake damage evaluation data for California. Applied Technology Council; 1985.
[45] Bramerini F, Di Pasquale G, Orsini G, Pugliese A, RomeoR, Sabetta F. Rischio sismico del territorio Italiano. 7° Convegno ANIDIS "L'Ingegneria Sismica in Italia"; 1995, p. 1099–108.
[46] Hazus M. Earthquake loss estimation methodology—technical and user manuals. Washington: Federal Emergency Management Agency; 1999.
[47] Dolce M, Marino M, Masi A, Vona M. Seismic vulnerability analysis and damage scenarios of Potenza. International Workshop on Seismic Risk and Earthquake Scenarios of Potenza; 2000.
[48] Basaglia A, Aprile A, Pilla F, Spacone E. Computed-aided risk assessment at urban scale model definition and validation on a case study. In: 7th European congress on computational methods in applied sciences and engineering, ECCOMAS Congress. National Technical University of Athens; 2016. p. 5977–86.
[49] Ferreira TM, H R, Vicente R, Costa A. Development of a Methodology for the seismic vulnerability assessment of existing RC buildings. In: 16th World Conference on Earthquake Engineering, 16WCEE; 2017a.
[50] Lungu D, Aldea A, Arion A, Vacareanu R, Petrescu F, Cornea T. WP1 report European distinctive features, inventory database and typology. RISK-UE project of the EC: an advanced approach to earthquake risk scenarios with applications to different European towns; 2001a.
[51] Ferreira TM, Vicente R, Da Silva JM, Varum H, Costa A. Seismic vulnerability assessment of historical urban centres: case study of the old city centre in Seixal, Portugal. Bull Earthq Eng 2013;11:1753–73.
[52] Cavaleri L, Di Trapani F, Ferrotto MF. A new hybrid procedure for the definition of seismic vulnerability in Mediterranean cross-border urban areas. Nat Hazards 2017;86:517–41.
[53] Cherif S-E, Chourak M, Abed M, Pujades L. Seismic risk in the city of Al Hoceima (north of Morocco) using the vulnerability index method, applied in Risk-UE project. Nat Hazards 2017;85:329–47.
[54] Lungu D, Aldea A, Arion A, Vacareanu R, Petrescu F, Cornea T. WP1 report: European distinctive features, inventory database and typology. RISK-UE project: an advanced approach to earthquake risk scenarios with applications to different European towns. Contract No. EVK4-CT-2000-00014; 2001b.
[55] Cherif S-E, Chourak M, Abed M, Douiri A. Potential seismic damage assessment of residential buildings in Imzouren City (Northern Morocco). Buildings 2018;8:179.
[56] Boughacha M, Ouyed M, Ayadi A, Benhallou H. Seismicity and seismic hazard mapping of northern Algeria: map of Maximum Calculated Intensities (MCI). J Seismolog 2004;8:1–10.
[57] Pelaez JA, Hamdache M, Casado CL. Updating the probabilistic seismic hazard values of northern Algeria with the 21 May 2003 M 6.8 Algiers earthquake included. Pure Appl Geophys 2005;162:2163–77.
[58] Kherroubi A, Deverchiere J, Yelles A, De Lepinay BM, Domzig A, Cattaneo A, et al. Recent and active deformation pattern off the easternmost Algerian margin, Western Mediterranean Sea: new evidence for contractional tectonic reactivation. Mar Geol 2009;261:17–32.
[59] Mourabit T, Elenean KA, Ayadi A, Benouar D, Suleman AB, Bezzeghoud M, et al. Neo-deterministic seismic hazard assessment in North Africa. J Seismolog 2014;18:301–18.
[60] Mahmoud B. Calculation of seismic vulnerability index for steel structures. Energy Procedia 2017;139:558–64.
[61] 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;8:326–41.
[62] FEMA. FEMA273, NEHRP Guidelines for the seismic rehabilitation of buildings. Federal Emergency Management Agency; 1997.
[63] FEMA. FEMA356, commentary for the seismic rehabilitation of buildings. Washington, DC: Federal Emergency Management Agency, 7; 2000.
[64] Zhang JZ, Jiang J, Li GQ. An improved consecutive modal pushover procedure for estimating seismic demands of multi-storey framed buildings. Struct Des Tall Special Build 2017;26:e1336.
[65] Miranda E. Approximate seismic lateral deformation demands in multistory buildings. J Struct Eng 1999;125:417–25.
[66] Gupta B, Kunnath SK. Adaptive spectra-based pushover procedure for seismic evaluation of structures. Earthquake Spectra 2000;16:367–92.
[67] Chopra AK, Chintanapakdee C. Evaluation of modal and FEMA pushover analyses: vertically "regular" and irregular generic frames. Earthquake Spectra 2004;20:255–71.
[68] Goel RK, Chopra AK. Evaluation of modal and FEMA pushover analyses: SAC buildings. Earthquake Spectra 2004;20:225–54.
[69] Maison BF, Bonowitz D. Discussion of seismic performance evaluation of pre-northridge steel frame buildings with brittle connections" by Kihak Lee and Douglas A. Foutch. J Struct Eng 2004;130:690–1.
[70] Zacharenaki AE, Fragiadakis M, Papadrakakis M. Reliability-based optimum seismic design of structures using simplified performance estimation methods. Eng Struct 2013;52:707–17.
[71] Fragiadakis M, Vamvatsikos D. Fast performance uncertainty estimation via pushover and approximate IDA. Earthquake Eng Struct Dyn 2010;39:683–703.
[72] Shafei B, Zareian F, Lignos DG. A simplified method for collapse capacity assessment of moment-resisting frame and shear wall structural systems. Eng Struct 2011;33:1107–16.
[73] Fiore A, Spagnoletti G, Greco R. On the prediction of shear brittle collapse mechanisms due to the infill-frame interaction in RC buildings under pushover analysis. Eng Struct 2016;121:147–59.
[74] Zameeruddin M, Sangle KK. Review on Recent developments in the performance-based seismic design of reinforced concrete structures. In: Structures. Elsevier; 2016. p. 119–33.
[75] Liu Y, Kuang J. Spectrum-based pushover analysis for estimating seismic demand of tall buildings. Bull Earthq Eng 2017;15:4193–214.
[76] Rahmani AY, Bourahla N, Bento R, Badaoui M. An improved upper-bound pushover procedure for seismic assessment of high-rise moment resisting steel frames. Bull Earthq Eng 2018;16:315–39.
[77] Chopra AK, Goel RK. A modal pushover analysis procedure for estimating seismic demands for buildings. Earthquake Eng Struct Dyn 2002;31:561–82.
[78] Kalkan E, Kunnath SK. Adaptive modal combination procedure for nonlinear static analysis of building structures. J Struct Eng 2006;132:1721–31.
[79] Antoniou S, Pinho R. Development and verification of a displacement-based adaptive pushover procedure. J Earthquake Eng 2004;8:643–61.
[80] Poursha M, Khoshnoudian F, Moghadam A. A consecutive modal pushover procedure for estimating the seismic demands of tall buildings. Eng Struct 2009;31:591–9.
[81] Poursha M, Khoshnoudian F, Moghadam A. A consecutive modal pushover procedure for nonlinear static analysis of one-way unsymmetric-plan tall building structures. Eng Struct 2011;33:2417–34.
[82] Khoshnoudian F, Kiani M. Modified consecutive modal pushover procedure for seismic investigation of one-way asymmetric-plan tall buildings. Earthquake Eng Eng Vib 2012;11:221–32.
[83] Kreslin M, Fajfar P. The extended N2 method considering higher mode effects in both plan and elevation. Bull Earthq Eng 2012;10:695–715.
[84] Brozovic M, Dolsek M. Envelope-based pushover analysis procedure for the approximate seismic response analysis of buildings. Earthquake Eng Struct Dyn 2014;43:77–96.
[85] Fajfar P. Capacity spectrum method based on inelastic demand spectra. Earthquake Eng Struct Dyn 1999;28:979–93.
[86] Poursha M, Samarin ET. The modified and extended upper-bound (UB) pushover method for the multi-mode pushover analysis of unsymmetric-plan tall buildings. Soil Dyn Earthquake Eng 2015;71:114–27.
[87] Belejo A, Bento R. Improved modal pushover analysis in seismic assessment of asymmetric plan buildings under the influence of one and two horizontal components of ground motions. Soil Dyn Earthquake Eng 2016;87:1–15.
[88] Bertero VV. Strength and deformation capacities of buildings under extreme environments. Struct Eng Struct Mech 1977;53:29–79.
[89] Bazzurro P, Cornell CA. Seismic hazard analysis of nonlinear structures. I: methodology. J Struct Eng 1994;120:3320–44.
[90] Bazzurro P, Cornell CA, Shome N, Carballo JE. Three proposals for characterizing MDOF nonlinear seismic response. J Struct Eng 1998;124:1281–9.
[91] Vamvatsikos D, Cornell CA. Incremental dynamic analysis. Earthquake Eng Struct Dyn 2002;31:491–514.
[92] Yun S-Y, Hamburger RO, Cornell CA, Foutch DA. Seismic performance evaluation for steel moment frames. J Struct Eng 2002;128:534–45.
[93] Lin T, Baker JW. Introducing adaptive incremental dynamic analysis: a new tool for linking ground motion selection and structural response assessment; 2013.
[94] Jalayer F, De Risi R, Manfredi G. Bayesian Cloud Analysis: efficient structural fragility assessment using linear regression. Bull Earthq Eng 2015;13:1183–203.
[95] Miano A, Jalayer F, Ebrahimia H, Prota A. Cloud to IDA: Efficient fragility assessment with limited scaling. Earthquake Eng Struct Dyn 2018;47:1124–47.

[96] Vamvatsikos D. Incremental dynamic analysis with two components of motion for a 3D steel structure. Proceedings of the 8th US National Conference on Earthquake Engineering. Citeseer; 2006.
[97] Kircil MS, Polat Z. Fragility analysis of mid-rise R/C frame buildings. Eng Struct 2006;28:1335–45.
[98] Asgarian B, Sadrinezhad A, Alanjari P. Seismic performance evaluation of steel moment resisting frames through incremental dynamic analysis. J Constr Steel Res 2010;66:178–90.
[99] Farsangi EN, Rezvani FH, Talebi M, Hashemi S. Seismic risk analysis of steel-mrfs by means of fragility curves in high seismic zones. Adv Struct Eng 2014;17:1227–40.
[100] Fanaie N, Ezzatshoar S. Studying the seismic behavior of gate braced frames by incremental dynamic analysis (IDA). J Constr Steel Res 2014;99:111–20.
[101] Nazri FM, Saruddin SNA. Seismic fragility curves for steel and reinforced concrete frames based on near-field and far-field ground motion records. Arabian J Sci Eng 2015;40:2301–7.
[102] Saruddin SNA, Nazri FM. Fragility curves for low-and mid-rise buildings in Malaysia. Procedia Eng 2015;125:873–8.
[103] Farsangi EN, Tasnimi AA. The influence of coupled horizontal–vertical ground excitations on the collapse margins of modern RC-MRFs. Int J Adv Struct Eng (IJASE) 2016;8:169–92.
[104] Farsangi EN, Yang T, Tasnimi A. Influence of concurrent horizontal and vertical ground excitations on the collapse margins of non-ductile RC frame buildings. Struct Eng Mech 2016;59:653–69.
[105] Gonzalez-Drigo R, Avila-Haro J, Pujades L, Barbat A. Non-linear static procedures applied to high-rise residential URM buildings. Bull Earthq Eng 2017;15:149–74.
[106] Fathieh A, Mercan O. Seismic evaluation of modular steel buildings. Eng Struct 2016;122:83–92.
[107] Sobhan M, Rofooei F, Attari NK. Buckling behavior of the anchored steel tanks under horizontal and vertical ground motions using static pushover and incremental dynamic analyses. Thin-Walled Struct 2017;112:173–83.
[108] Moazam AM, Hasani N, Yazdani M. Incremental dynamic analysis of small to medium spans plain concrete arch bridges. Eng Fail Anal 2018;91:12–27.
[109] Azizan NZN, Majid TA, Nazri FM, Maity D, Abdullah J. Incremental Dynamic Analysis of Koyna Dam under Repeated Ground Motions. In: IOP Conference Series: Materials Science and Engineering. IOP Publishing; 2018,p. 012021.
[110] Kildashti K, Mirzadeh N, Samali B. Seismic vulnerability assessment of a case study anchored liquid storage tank by considering fixed and flexible base restraints. Thin-Walled Struct 2018;123:382–94.
[111] Roca P. Structural analysis of historical constructions: possibilities of numerical and experimental techniques, International Center for Numerical Methods in Engineering; 2011.
[112] Shome N. Probabilistic seismic demand analysis of nonlinear structures. Stanford University; 1999.
[113] Krawinkler H, Medina R, Alavi B. Seismic drift and ductility demands and their dependence on ground motions. Eng Struct 2003;25:637–53.
[114] Bakhshi A, Asadi P. Probabilistic evaluation of seismic design parameters of RC frames based on fragility curves. Scientia Iranica 2013;20:231–41.
[115] Silva V, Crowley H, Varum H, Pinho R, Sousa R. Evaluation of analytical methodologies used to derive vulnerability functions. Earthquake Eng Struct Dyn 2014;43:181–204.
[116] Muntasir Billah A, Shahria Alam M. Seismic fragility assessment of highway bridges: a state-of-the-art review. Struct Infrastruct Eng 2015;11:804–32.
[117] Colapietro D, Netti A, Fiore A, Fatiguso F, Marano GC. On the definition of seismic recovery interventions in rc buildings by non-linear static and incremental dynamic analyses. Int J Mech 2014;8:216–22.
[118] Ozmen H, Inel M, Meral E, Bucakli M. Vulnerability of low and mid-rise reinforced concrete buildings in Turkey. In: Proceedings of the 14th European conference on earthquake engineering, Ohrid, Macedonia; 2010.
[119] Polese M, Verderame GM, Mariniello C, Iervolinoe I, Manfredi G. Vulnerability analysis for gravity load designed RC buildings in Naples–Italy. J Earthquake Eng 2008;12:234–45.
[120] Ahmad N, Crowley H, Pinho R. Analytical fragility functions for reinforced concrete and masonry buildings and buildings aggregates of Euro-Mediterranean regions -. UPAV methodology; 2011.
[121] Pnevmatikos NG, Papagiannopoulos GA, Papavasileiou GS. Fragility curves for mixed concrete/steel frames subjected to seismic excitation. Soil Dyn Earthquake Eng 2019;116:709–13.
[122] Saloustros S, Pela L, Contrafatto FR, Roca P, Petromichelakis I. Vulnerability assessment of monumental masonry structures including uncertainty. Structural Analysis of Historical Constructions. Springer; 2019.
[123] Özel AE, Guneyisi EM. Effects of eccentric steel bracing systems on seismic fragility curves of mid-rise R/C buildings: A case study. Struct Saf 2011;33:82–95.
[124] Hajimehrabi H, Behnamfar F, Samani AK, Goudarzi MA. Fragility curves for baffled concrete cylindrical liquid-storage tanks. Soil Dyn Earthquake Eng 2019;119:187–95.
[125] Beillic 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;12:359–74.
[126] Babic A, Dolsek M. Seismic fragility functions of industrial precast building classes. Eng Struct 2016;118:357–70.
[127] Banerjee AK, Pramanik D, Roy R. Seismic structural fragilities: Proposals for improved methodology per spectral matching of accelerogram. Eng Struct 2016;111:538–51.
[128] Casotto C, Silva V, Crowley H, Nascimbene R, Pinho R. Seismic fragility of Italian RC precast industrial structures. Eng Struct 2015;94:122–36.
[129] Senel SM, Kayhan AH. Fragility based damage assesment in existing precast industrial buildings: a case study for Turkey. Struct Eng Mech 2010;11:39.
[130] Ahmad N, Ibrahim A, Alam S. Analytical seismic fragility curves for reinforced concrete wall pier using shape memory alloys considering maximum drift. In: MATEC Web of Conferences. EDP Sciences; 2019, p. 04001.
[131] Batalha N, Rodriques H, Varum H. Seismic performance of RC precast industrial buildings—learning with the past earthquakes. Innovative Infrastruct Solut 2019;4:4.
[132] Shinozuka M, Feng MQ, Kim H-K, Kim S-H. Nonlinear static procedure for fragility curve development. J Eng Mech 2000;126:1287–95.
[133] Siqueira GH, Sanda AS, Paultre P, Padgett JE. Fragility curves for isolated bridges in eastern Canada using experimental results. Eng Struct 2014;74:311–24.
[134] Alessandri S, Giannini R, Paolacci F. A new method for probabilistic aftershock risk evaluation of damaged bridge. Proceedings of the conference 3rd ECCOMAS thematic conference on computational methods in structural dynamics and earthquake engineering, 2011.
[135] Long X, Xie Z, Fan J, Miao Y. Convex model-based calculation of robust seismic fragility curves of isolated continuous girder bridge. Bull Earthq Eng 2018;16:155–82.
[136] Segura R, Bernier C, Monteiro R, Paultre P. On the seismic fragility assessment of concrete gravity dams in eastern Canada. Earthquake Spectra 2018.
[137] D'amico M, Buratti N. Observational seismic fragility curves for steel cylindrical tanks. J Press Vessel Technol 2019;141:010904.
[138] Vona M. Fragility curves of existing RC buildings based on specific structural performance levels. Open J Civil Eng 2014;4:120.
[139] Anvarsamarin A, Rofooei FR, Nekooei M. Soil-structure interaction effect on fragility curve of 3D models of concrete moment-resisting buildings. Shock Vib 2018.
[140] Tajammolian H, Khoshnoudian F, Rad AR, Loghman V. Seismic Fragility assessment of asymmetric structures supported on TCFP Bearings subjected to near-field earthquakes. Structures. Elsevier; 2018, p. 66–78.
[141] Nazari YR, Saatcioglu M. Seismic vulnerability assessment of concrete shear wall buildings through fragility analysis. J Build Eng 2017;12:202–9.
[142] Pekelnicky R, Engineers SD, Chris Poland S. ASCE 41-13: Seismic evaluation and retrofit rehabilitation of existing buildings. Proceedings, SEAOC 2012 Convention; 2012.
[143] Karapetrou S, Fotopoulou S, Pitilakis K. Seismic vulnerability of RC buildings under the effect of aging. Procedia Environ Sci 2017;38:461–8.
[144] Banazadeh M, Ghanbari A. Seismic performance assessment of steel moment-resisting frames equipped with linear and nonlinear fluid viscous dampers with the same damping ratio. J Constr Steel Res 2017;136:215–28.
[145] Rota M, Penna A, Magenes G. A methodology for deriving analytical fragility curves for masonry buildings based on stochastic nonlinear analyses. Eng Struct 2010;32:1312–23.
[146] Oropeza M, Michel C, Bigler M, Lestuzzi P. New analytical fragility curves for existing URM buildings in regions with moderate seismicity. In: 8th International Masonry Conference, Dresden, Germany; 2010.
[147] Ceran H. Seismic vulnerability of masonry structures in Turkey. Master Thesis. Department of Civil Engineering of the Middle East Technical; 2010.
[148] Pitilakis K. Systemic seismic vulnerability and risk analysis for buildings, lifeline networks and infrastructures safety gain; 2011.
[149] Pagnini LC, Vicente R, Lagomarsino S, Varum H. A mechanical model for the seismic vulnerability assessment of old masonry buildings. Earthq Struct 2011;2:25–42.
[150] Ahmad N, Crowley H, Pinho R. Analytical fragility functions for reinforced concrete and masonry buildings and building aggregates of Euro-Mediterranean regions. Department of Structural Mechanics, University of Pavia. WP3-Task, 3; 2011a.
[151] Akansel VH, Yakut A, Gulkan P. Fragility of shear wall buildings with torsional Irregularity. In: Proceedings of the 15th world conference on earthquake engineering (WCEE). Lisbon, Portugal; 2012.
[152] Fardis MN, Papailia A, Tsionis G. Seismic fragility of RC framed and wall-frame buildings designed to the EN-Eurocodes. Bull Earthq Eng 2012;10:1767–93.
[153] Bilgin H. Fragility-based assessment of public buildings in Turkey. Eng Struct 2013;56:1283–94.
[154] Karantoni F, Tsionis G, Lyrantzaki F, Fardis MN. Seismic fragility of regular masonry buildings for in-plane and out-of-plane failure. Earthq Struct 2014;6:689–713.
[155] Pitilakis K, Crowley H, Kaynia A. SYNER-G: typology definition and fragility functions for physical elements at seismic risk. Geotechn, Geol Earthq Eng, 2014;27.
[156] Tsionis G, Fardis MN. Seismic fragility curves for reinforced concrete buildings and bridges in Thessaloniki. 2nd European Conference on Earthquake Engineering and Seismology, Istanbul; 2014.
[157] Silva V, Crowley H, Varum H, Pinho R. Seismic risk assessment for mainland Portugal. Bull Earthq Eng 2015;13:429–57.
[158] Simooes A, Milosevic J, Meireles H, Bento R, Cattari S, Lagomarsino S. Fragility curves for old masonry building types in Lisbon. Bull Earthq Eng 2015;13:3083–105.
[159] Del Gaudio C, Ricci P, Verderame G, Manfredi G. Development and urban-scale application of a simplified method for seismic fragility assessment of RC buildings. Eng Struct 2015;91:40–57.

[160] Hancilar U, Cactı E. Fragility functions for code complying RC frames via best correlated IM–EDP pairs. Bull Earthq Eng 2015;13:3381–400.
[161] Shahidi SG, Pakzad SN, Ricles JM, Martin JR. Assessment of the 2011 Virginia earthquake damage and seismic fragility analysis of the Washington Monument. Earthquake Spectra 2016;32:2399–423.
[162] Villar-Vega M, Silva V, Crowley H, Yepes C, Tarque N, Acevedo AB, et al. Development of a fragility model for the residential building stock in South America. Earthquake Spectra 2017;33:581–604.
[163] Dabaghi M, Saad G, Allhassaania N. Seismic collapse fragility analysis of reinforced concrete shear wall buildings. Earthquake Spectra 2018.
[164] Amirihormozaki E, Pekcan G, Itani A. Analytical fragility functions for horizontally curved steel I-girder highway bridges. Earthquake Spectra 2015;31:2235–54.
[165] Lovon H, Tarque N, Silva V, Yepes-Eestrada C. Development of fragility curves for confined masonry buildings in Lima, Peru. Earthquake Spectra 2018.
[166] Hoult R, Goldsworthy H, Lumantarna E. Fragility functions for RC shear wall buildings in Australia. Earthquake Spectra 2018.
[167] Maison B, Mcdonald B. Fragility curves for residential masonry chimneys. Earthquake Spectra 2018;34:1001–23.
[168] Nazri FM, Miari MA, Kassem MM, Tan C-G, Farsangi EN. Probabilistic evaluation of structural pounding between adjacent buildings subjected to repeated seismic excitations. Arabian J Sci Eng 2018:1–15.
[169] Wijayanti E, Kristiawan S, Purwanto E, Sangadji S. Seismic vulnerability of reinforced concrete building based on the development of fragility curve: A case study. Applied Mechanics and Materials. Trans Tech Publ; 2016, p. 252–58.
[170] Mccrum D, Amato G, Suhail R. Development of seismic fragility functions for a moment resisting reinforced concrete framed structure. Open Constr Build Technol J 2016;10.
[171] Pejovic J, Jankovic S. Seismic fragility assessment for reinforced concrete high-rise buildings in Southern Euro-Mediterranean zone. Bull Earthq Eng 2016;14:185–212.
[172] Negulescu C, Ulrich T, Baills A, Seyedi D. Fragility curves for masonry structures submitted to permanent ground displacements and earthquakes. Nat Hazards 2014;74:1461–74.
[173] Ibrahim YE, EL-Shami MM. Seismic fragility curves for mid-rise reinforced concrete frames in Kingdom of Saudi Arabia. IES J Part A: Civil Struct Eng 2011;4:213–23.
[174] Molina-Palacios S. State of the art in seismic vulnerability; 2007.
[175] Nazari FM. Seismic fragility assessment for buildings due to earthquake excitation. Springer; 2018.
[176] Ortega J, Vasconcelos G, Rodrigues H, Correia M. A vulnerability index formulation for the seismic vulnerability assessment of vernacular architecture. Eng Struct 2019;197:109381.
[177] Mosoarca M, Onescu I, Onescu E, Azap B, Chieffo N, Szitar-Sirbu M. Seismic vulnerability assessment for the historical areas of the Timisoara city, Romania. Eng Fail Anal 2019;101:86–112.
[178] Aguilar-Meléndez A, Pujades LG, Barbat AH, Ordaz MG, de la Puente J, Lantada N, et al. A probabilistic approach for seismic risk assessment based on vulnerability functions. Application to Barcelona. Bull Earthq Eng 2019;17 (4):1863–90.
[179] Ortega J, Vasconcelos G, Rodrigues H, Correia M, Ferreira TM, Vicente R. Use of post-earthquake damage data to calibrate, validate and compare two seismic vulnerability assessment methods for vernacular architecture. Int J Disaster Risk Reduct 2019;39:101242.

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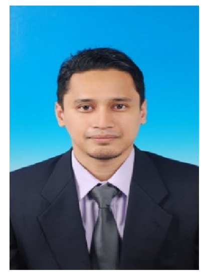

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