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Research Article:

Seismic vulnerability assessment of unreinforced masonry single story buildings in Sulaymaniyah city

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Abstract

Unreinforced masonry (URM) buildings are among the most vulnerable structural types in seismic regions, particularly in developing cities where construction quality and material properties vary significantly. Although seismic vulnerability of URM buildings has been widely investigated worldwide, studies incorporating locally tested material properties and calibrated analytical fragility models at the city scale remain limited. The novelty of this research is to addresses this gap by developing a comprehensive seismic vulnerability assessment framework specifically for URM buildings in Sulaymaniyah City, Iraq. This study begins with a review of established empirical and analytical techniques used globally for seismic vulnerability assessment, outlining their strengths, limitations, and typical applications. In Sulaymaniyah City, a field survey of 4,724 buildings based on the EMS-98 classification showed that most buildings fall into vulnerability classes B and C under pessimistic and optimistic scenarios, respectively, and are predominantly constructed using hollow concrete blocks. A representative URM building was analyzed through nonlinear static analysis using the Equivalent Frame Method implemented in TREMURI, with material properties obtained from experimental tests (mean compressive strength ≈ 3.0 MPa and elastic modulus ≈ 2,400 MPa). The results revealed lower lateral strength and stiffness in the x-direction, leading to higher probabilities of slight damage at low PGA levels (around 0.05g) compared to the more resilient y-direction, highlighting the importance of considering directional behavior and local material properties in seismic risk mitigation.

1. Introduction

Masonry is one of the oldest and most widely used construction materials, favored for its low cost, ease of construction, minimal labor requirements, environmental friendliness, and reliance on locally available resources [1]. In many developing countries, it remains the most common choice for low-income housing [2]. Despite these advantages, unreinforced masonry (URM) buildings are highly vulnerable to earthquakes [3]. Their mechanical behavior is strongly influenced by the properties of the units, mortar, and connections, while construction deficiencies and geometric irregularities often contribute to local or out-of-plane collapse mechanisms [3]. Most existing URM buildings were not designed to resist seismic loads, being originally intended only to carry vertical loads [4]. As a result, they exhibit very limited resistance to horizontal seismic forces [5]. Earthquakes are severe events that release significant energy, produce strong ground shaking and imposing critical horizontal demands on structures [6] In recent decades, the increasing frequency of seismic events has further emphasized the urgent need to evaluate the safety of existing URM buildings. Many of these buildings were constructed without seismic design considerations [7] and they may have changed significantly over time [8]. which increases their overall vulnerability [3]. Consequently, assessing the seismic safety of masonry buildings has become a priority, both to reduce risk for the population and to preserve architectural heritage. Seismic risk assessment is commonly described in terms of three key components: hazard, exposure, and vulnerability. Hazard reflects the probability of ground shaking exceeding a certain level at a given location, exposure relates to the properties and population at risk, and vulnerability denotes the structural capacity to resist damage [5]. Within this framework, national authorities increasingly require seismic vulnerability studies of URM structures to support risk mitigation and resilience planning. Methods for seismic vulnerability assessment are generally classified into three levels-building-level, local, and large-scale approaches [9] depending on the scope of application, as shown in figure 1. Two main methodological families are typically used: empirical and analytical approaches. Empirical approaches rely on post-earthquake damage data and classify buildings according to typology, such as structural system, materials, and number of stories [10 - 13]. While suitable for large-scale studies, they often face limitations related to data availability and assumptions. Analytical approaches, in contrast, are based on numerical modeling structural behaviour under seismic actions [11, 12] Although more detailed and accurate, they are computationally intensive and require reliable input data on material and geometry. Hybrid methods that combine the strengths of both approaches are also increasingly applied. In Sulaymaniyah City, the majority of residential buildings are constructed using unreinforced masonry (URM) and therefore potentially highly vulnerable to seismic activity. Yet, few studies have focused on modeling these structures using locally obtained material properties, which are essential for accurately capturing their seismic response. To address this gap, the present study first reviews the main approaches to URM vulnerability assessment. It then develops an analytical model of a representative local building using experimental data to evaluate its seismic performance. The results are compared with findings from the regional literature to validate the methodology and highlight similarities and differences in seismic behavior. Ultimately, this study provides new insights into the seismic vulnerability of local URM buildings in Sulaymaniyah, offering a basis for future research and informing strategies for retrofitting and risk reduction.

2. State-of-the-Art Review on URM Seismic Performance.

To illustrate the level of seismic demand that an earthquake exerts on a structure, a vulnerability assessment must be conducted for a specific ground motion characterization. The chosen parameter needs to be able to link the building damage and ground motion. Peak ground acceleration (PGA) and macro seismic intensity have historically been utilized, but more recent theories have connected the seismic sensitivity of structure to reaction spectra derived from ground movements. Analytical approaches connect this to the limit-state mechanical attributes of the buildings, including inter-story drift capacity, whereas Empirical vulnerability methods use damage scales derived from observations to provide post-earthquake damage statistics [10].

2. 1 Empirical Approach:

Large-scale seismic vulnerability assessments of buildings were first initiated in the 1970s using empirical methods based on macroseismic intensities [10,12] and remain widely applied due to their reliability [3]. These methods, termed “damage-motion relationships” [10], include two main approaches: Damage Probability Matrices (DPM) and the Vulnerability Index Method (VIM). DPMs, considered a “direct” method use historical earthquake data to correlate structural typologies with expected damage levels [13]. In contrast, the “indirect” VIM quantifies vulnerability through weighted structural parameters, later calibrated with observed damage [10,14]. One of key implementation is the EMS-98 macroseismic method and illustrated in table 1 [14] which classifies buildings into six vulnerability classes (A to F) based on their structural characteristics [11,15]. Various case studies across Europe have applied macro-seismic and index-based methods to assess the vulnerability of different building typologies. In Zagreb, Croatia, earthquakes in March 2020 highlighted the high vulnerability of masonry buildings with deformable floors, shown through fragility curves developed for 66 structural units using past damage data and EMS-98 intensity measures [12]. Similarly, in Arsita, Italy, seismic behavior varied among structural units in a small aggregate, with the highest vulnerability found in corner units and a global vulnerability index of 0.53 [3]. In Bosnia and Herzegovina, a large-scale assessment of 2,933 buildings in Sarajevo, Banja Luka, and Tuzla revealed that unreinforced masonry (URM) with flexible floors was the most vulnerable, while confined masonry performed better [11]. Structural features such as floor stiffness, construction quality, and retrofitting strongly influence vulnerability, rigid slabs promote box-like behavior, enhancing resistance, whereas flexible slabs increase seismic risk [16]. Unreinforced masonry and wooden structures remain particularly susceptible to collapse during strong earthquakes, necessitating extensive strengthening [15]. Furthermore, construction period and structural classification significantly affect post-earthquake usability, especially in historic urban areas, underlining the importance of tailored mitigation strategies [17].

2.2 Analytical/mechanical Approach

Analytical methods assess seismic vulnerability by simulating structural response based on physical and mechanical principles, offering more transparent and adaptable evaluations than empirical approaches [14,10]. These methods are capable of capturing both in-plane and out-of-plane behaviors of masonry buildings, using various types of static and dynamic analyses to express damage through parameters like displacement and drift [15]. Moreover, analytical approaches allow calibration to specific building types and hazard levels without relying on past earthquake data. Numerical modeling, especially the Finite Element Method (FEM), is widely used to simulate masonry structures. FEM can adopt micro- or macro-modeling strategies depending on the required accuracy and scale [18]. Macro-modeling is typically applied to entire buildings, while meshing techniques divide the structure into 1D, 2D, or 3D elements [19]. Another effective tool is the Equivalent Frame Method, which models walls using macro-elements like piers and spandrels connected by rigid links, focusing on typical in-plane failure modes such as diagonal cracking or shear [20,21]. This method simplifies masonry complex behavior while capturing key damage mechanisms, making it suitable for evaluating existing masonry under seismic loads [22], as illustrated in figure 2. For nonlinear analyses, three-dimensional finite element analysis is frequently too time-consuming and complicated because of difficulties like numerical instability and the requirement for numerous input parameters. While equivalent frame modeling (EFM) is frequently employed in both research and practice and represents one of the most commonly used approaches. The EFM consider the walls as an idealized frame [4]. Nonlinear analysis methods provide a more realistic and economical approach to evaluating masonry structures, capturing behaviors such as cracking and reduced load capacity due to low tensile of masonry strength [23]. Unlike linear elastic methods, nonlinear techniques such as pushover and time history analyses are essential for simulating seismic performance [24 ,25]. Pushover analysis, favored for its simplicity and computational efficiency, applies incrementally increasing lateral loads until collapse and is often used with procedures including as the N2 method and capacity spectrum method [18,26]. Time history analysis, though computationally complex, offers more accurate results by applying recorded ground motions to simulate real seismic response [25,18]. Although nonlinear dynamic analysis time-history is the most accurate method, it is infrequently applied in engineering practice due to its high computational demands. Therefore, the proposed approach adopts nonlinear static (pushover) analysis as a practical alternative. Several studies have investigated analytical approaches for assessing URM buildings under seismic loads, highlighting the strengths and limitations of various modeling techniques. Betti et al. [27] compared finite element (FEM) and macro-element (ME) models for a prototype with flexible diaphragms. While the FE Model better captured damage patterns, the ME model overestimated displacements by up to 85% and showed spectral acceleration errors of 20–80%, despite predicting collapse loads well. Similarly, Pacella et al. [28] used EFM to study the seismic response of brick walls, noting that spandrels increased base shear capacity while reducing ductility. Comparison with 2D-shell models showed large ultimate displacements in the latter due to differing tensile resistance assumptions. In the same study, the flange effect was examined using a case study in Pizzoli demonstrating that stronger orthogonal wall connections significantly increase base shear and stiffness. Complementing this by [29] analyzed a two-story irregular URM structure using three models—DIANA, 3DMacro, and TREMURI—validated against shaking table tests, Figure 3. The continuum model (DIANA) showed the best match with only 4% average deviation, while 3DMacro overestimated, and TREMURI was overly conservative. These findings emphasized the importance of torsional effects and wall-slab connections are critical, influencing failure modes like rocking and diagonal shear. The EFM is particularly advantageous for large-scale analysis of masonry buildings because it simplifies complex wall behavior into macro-elements (piers and spandrels), allowing for efficient and relatively accurate simulation of in-plane failure mechanisms. It balances computational efficiency with sufficient detail, making it well-suited for performance-based seismic assessments, especially when resources or data availability are limited [29]. Avila-Haro et al. [30] compared nonlinear static procedures NSPs with Incremental Dynamic Analysis (IDA), showing that while NSPs overestimated displacements, the 10% fit method yielded closer results to IDA by better capturing initial stiffness. Recent research has evaluated the applicability of (NSPs) for irregular URM buildings. Marino et al. [31] found that uniform and SRSS load patterns per-formed best and Giordano et al. [32] showed that pushover analysis underestimated displacement on the flexible side by about 20%, suggesting improved accuracy when shifting lateral loads toward the flexible side. Using the Equivalent Frame Model, Sonekar and Bakre [33] found that uniform load patterns led to higher strength estimates, while mode and parabolic patterns increased capacity by about 15%. Spectral analysis revealed 64% lower displacement in the strong direction, but 27% higher spectral acceleration. Chieffo et al. [3] compared the Empirical vulnerability index and mechanical approaches for a masonry building, finding the index method suitable for urban-scale use but becomes less accurate when PGA > 0.3g. The mechanical method (macro-element model) showed better damage prediction, especially in the Y direction. [3] further found that the EFM gave vulnerability index values 16% higher in X and 36% in Y than the macro-seismic method, offering a more conservative assessment. Similarly, Moretić et al. [6] noted that while both methods aligned for moderate damage (<5%), the macro-seismic method underestimated severe damage by up to 16.5%, especially in the x- direction, whereas the analytical method provided more direction-specific accuracy as represented in figure 4. [5].

3. Case study in Sulaymaniyah city

Sulaymaniyah located in the eastern part of Iraq's Kurdistan Region, has seen rapid urban growth due to economic development and population increase. A seismic vulnerability assessment by Hasan et al. [34] of 4,724 buildings was conducted using the EMS-98 classification. Results showed that 55% of the buildings were non-engineered, making them highly vulnerable to earthquakes. Single-family homes were the most common, with M6-type structures—unreinforced masonry with reinforced concrete floors accounting for 74.9% of the sample. The buildings were classified according to vulnerability under both optimistic and pessimistic scenarios. In the optimistic case, most buildings fell into class C, while in the pessimistic scenario, class B was dominant, reflecting concerns over construction quality. Based on this survey, a representative house was selected for detailed numerical modeling as the case study [34].

3.1 Development of the Numerical Model

The case study focuses on an URM building typical of those found in Sulaymaniyah and across the Kurdistan Region. The EFM implemented in the TREMURI software, was employed for structural analysis, considering the stiffness and in-plane strength of masonry components. In this approach, the nonlinear behavior of piers and spandrels is modeled using a macro-element formulation. Each microelement representing either a pier or a spandrel is subdivided into three segments (as illustrated in figure 5, which each segment defined by eight in-plane degrees of freedom. These include axial displacements (w_i and w_j), lateral displacements (u_i and u_j), and rotations (φ_i and φ_j) at the element ends (i and j), along with two rigid-body displacements (w_e and φ_i) located in the central segment.[35] Coupled axial and rocking interactions are captured at the macro-element's top and bottom inter-faces, whereas the middle portion primarily represents shear behavior. The axial-rocking behavior captures the masonry's limited compressive capacity, while the shear response incorporates degradation in strength and stiffness governed by an internal damage variable. This formulation enables a realistic simulation of in-plane seismic performance, accounting for progressive deterioration under increasing deformation demands [35].

3.2 Experimental program

To obtain material properties and input to the software experimental work performed as shown in figure 6. The tested walls were built using hollow concrete block masonry with a 1:2.75 cement-sand mortar in a stretcher bond pattern, which represents the typical masonry construction method in Iraq. The blocks were arranged in a staggered layout to prevent the formation of continuous vertical joints. Individual unit blocks had average dimensions of 401 × 200 × 198 mm. The unit block tests showed an average compressive strength of 7.03 MPa with a standard variation of 1.3 MPa. The mechanical properties of mortar were evaluated through compression and flexural tests. Compressive strength was determined in accordance with ASTM C109 using 50 × 50 × 50 mm cubes, yielding an average value of 33.86 MPa at 28 days. Flexural strength was measured in accordance with ASTM C348 on 40 × 40 × 160 mm prisms, with an average result of 7.05 MPa after 28 days. Average compressive strength and modulus of elasticity of masonry were (3.01 MPa and 2,400 MPa) respectively as determined from tests conducted on masonry prisms, 599 mm long x 629 mm high x 199 mm thick according to ASTM C1314 [2012][36], as shown in Figure 6.a. Shear strength was (0.46 MPa) calculated from diagonal compression test on masonry prisms that were 600 mm long x 630 mm high x 199 mm thick ASTM E 519-02 [2002], as shown in Figure 6.b. and modulus of rigidity was (1000 MPa). The selected building is a one-story structure with a 90 m² area, irregular in plan, with rigid diaphragms, 20 cm wall thickness, and a story height of 3.0 m. Figure 7.

3.3 Description of the Pushover Analysis

Pushover analysis involves applying constant gravity loads and gradually increasing horizontal forces. Two vertical load distributions are considered: a uniform pattern proportional to the building's mass, and a modal pattern based on the first mode shape obtained from elastic analysis. These loads are applied at the center of mass on each floor, and accidental eccentricity is included to address uncertainties in mass center location [36]. At each load increment, internal forces are redistributed based on element equilibrium, and stiffness is reduced as elements enter the plastic range. Ductility was monitored using maximum drift limits: 0.004 for shear failure and 0.008 for bending failure [37]. Floors were assumed to act as rigid diaphragms in the 3D model, reflecting their in-plane stiffness. While the software assigns infinite in-plane axial stiffness to these diaphragms, the real slab mass is properly accounted for in the analysis. Turnsek-Cacovic constitutive law was employed diagonal cracking is usually the dominant failure mode for existing unreinforced masonry structures [38]. Material properties obtained from on-site testing (table 2) were used as input for the numerical model. Based on the level of knowledge attained, a confidence factor of 1 was determined. Elastic response spectra for acceleration are computed based on Iraqi seismic code [39] with consideration of soil type C as it is the most widely distributed soil type in Sulaymaniyah City based on three investigation methods reported in [40] and importance factor is I = 1. In total, 24 pushover analyses are conducted in both directions as representing in figure 8, with two load distributions. Capacity curves, illustrating the relationship between the roof displacement at a specified control node and the base shear force obtained by the seismic analysis. The control node was situated on the top floor. The capacity curves obtained from each of the 24 analyses are shown in Fig.8, kilonewtons (kN) on the y-axis and displacement in (mm) on the x-axis. The structural capacity was evaluated based on limit states after analyzing the response. Parameters for equivalent SDOF (Single Degree Of Freedom) systems, derived using the equivalent energy principle during bilinearization, are listed in Table 3 and used to determine the target displacement. According to Eurocode 8 (Appendix B) [38], the target displacement is calculated, using the procedure applicable to buildings with short natural periods, as the structure's period is less than the characteristic period T_C. Such short-period buildings do not follow the equal displacement rule, unlike those with medium or long natural periods [23, 36].

3.4 Damage States

Defining structural damage levels is essential for developing fragility curves. These levels, often based on displacement, indicate when a building can no longer perform its intended function. While fixed drift thresholds are commonly used, more accurate results are achieved by considering a structure's specific strength and ductility. Based on the recommendations of Milutinovic and Trendafiloski [41], this study classifies damage into five states: slight, moderate, extensive, very heavy, and collapse, as summarized in Table 4. Yield and ultimate displacements required for this classification were obtained from pushover analysis d_y=2.5mm and d_u=8.96mm for x-direction and d_y=1.20mm and d_u=2.19mm for y-direction. The yield displacement (d_y) indicates the onset of nonlinear behavior and the transition from elastic to inelastic response, while the ultimate displacement (d_u) represents the maximum deformation capacity before significant strength degradation or collapse.

3.5 Development of fragility curves

The statistical characterization of building response is represented using a lognormal distribution, following the methodology proposed by Vamvatsikos and Allin[42]. This allows the computation of the probability of exceeding predefined damage states, when plotted against an intensity measure such as Peak Ground Acceleration (PGA), yields a fragility curve. In this study, fragility curves were developed using a two-parameter lognormal distribution:

$$f(x) = P(d \geq D) = \Phi\left(\frac{ln(x)-ln(\mu)}{\beta}\right)$$

Where, Φ is the standard normal cumulative distribution function, x is the distributed intensity measure (e.g., PGA), D is the damage state, and μ and β are the mean and standard deviation of the natural logarithm of the intensity measures, respectively. Within the framework of pushover analysis, these parameters were estimated from PGA values corresponding to each damage state. To construct fragility curves, twelve PGA levels (0.05g to 2.0g) in the x-y direction used, can be seen in Figure 9. PGA was selected as the intensity measure due to its suitability for short-period structures, ease of derivation, and widespread availability in seismic hazard maps. The resulting PGA-based fragility curves indicate that single-story unreinforced masonry (URM) buildings in the Kurdistan Region are particularly vulnerable to slight damage even under low seismic intensities. Moderate damage is likely in areas where PGA exceeds 0.05g, while extensive and very heavy damage may occur in the eastern Region of Kurdistan, where PGA surpasses 0.15g. However, collapse is not anticipated within the typical seismic intensity range for the region. The fragility curves clearly reveal directional vulnerability in the seismic response of the analyzed building. In the x-direction, the curves are steep and shifted to lower PGA values, with slight and moderate damage being exceeded at 0.05g to 1.0 g, and very heavy damage initiating beyond 0.15 g. This indicates a high susceptibility to damage under relatively low ground motion intensities, likely due to lower lateral stiffness, larger openings, or a lack of sufficient structural walls in that direction. The x-direction also corresponds to the shorter side of the building, where seismic forces tend to concentrate. In contrast, the y-direction curves are flatter and shifted toward higher PGA values (0.6 g –1.0 g), with minimal probability of collapse even at very high intensities, suggesting a much stiffer and stronger behavior. This improved performance may be attributed to the presence of more continuous walls, shear resisting elements, or inherently stiffer geometry along the y-axis. These findings underscore the need for targeted retrofitting strategies in the x-direction to enhance the building's overall seismic resilience and highlight the importance of evaluating structural performance in both orthogonal directions for comprehensive seismic assessment. The fragility curves obtained in this study are compared with those presented in a previous regional assessment of low-rise URM buildings in the Kurdistan Region. [42] adopted an Incremental Dynamic Analysis (IDA) approach using 59 unscaled ground motions to develop fragility curves for a typical URM house with dimensions of (10 × 15 m and heights of 3.0 m and 6.0 m) constructed with solid concrete block walls displayed in figure 10.a. In that study, fragility curves were developed in terms of PGA expressed in m/s² and presented only for the x-direction Figure 10.b. In the reference study [43], slight damage appeared at 0.63 mm and collapsed at 12.66 mm, whereas in present case study slight damage already occurred at 1.48 mm and collapse followed at only 7.16 mm. This means your building reaches the onset of damage later than the reference (slight damage at 1.48 mm vs. 0.63 mm), but it progresses much faster to collapse, showing a narrower safety margin and lower ductility. The reason lies in the weaker experimentally tested materials (E = 2,400 MPa) and the less favorable narrow plan shape of your building, compared to the stronger assumed materials (E = 4,350 MPa) and more regular geometric configuration reference. As a result, present fragility curves show higher vulnerability at lower PGA levels, confirming that the studied building is more fragile and prone to rapid degradation once damage begins.

4. Conclusion

This study investigated the seismic vulnerability of unreinforced masonry buildings in Sulaymaniyah city through a detailed analytical assessment of a representative single-story structure. The Equivalent Frame Model (EFM) was implemented to perform a detailed nonlinear static (pushover) analysis of a representative single-story URM building. The building exhibited notable directional seismic vulnerability. Pushover analysis results showed that the lateral capacity in the x-direction was significantly lower (F^*y = 232 kN) compared to the y-direction (F^*y = 1,128 kN), a difference of over 79%, likely due to fewer structural walls and larger openings. The corresponding ultimate displacements were 7.16 mm and 2.93mm, respectively, indicating significantly lower ductility in the x-direction. Fragility curves further confirmed that slight and moderate damage states are exceeded at very low seismic intensities (PGA ≈ 0.05g) in the x-direction, while the y-direction remains more resistant even at PGA levels of 0.6g or higher. Comparison with a regional reference study showed that, although the analyzed building experienced a delayed onset of initial damage, it progressed more rapidly toward collapse, reflecting higher vulnerability and limited deformation capacity.

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تقييم قابلية المباني ذات الطابق الواحد المبنية من الطوب غير المسلح للتأثر بالزلازل في مدينة السليمانية

المخلص

تُعدّ المباني المبنية من الطوب غير المسلح من بين أكثر أنواع المباني عرضةً للزلازل في المناطق المعرضة للزلازل، لا سيما في المدن النامية حيث تتباين جودة البناء وخصائص المواد بشكل كبير. ورغم أن هشاشة هذه المباني قد خضعت لدراسات واسعة النطاق عالمياً، إلا أن الدراسات التي تُدمج خصائص المواد المختبرة محليًا ونماذج الهشاشة التحليلية المُعايرة على مستوى المدينة لا تزال محدودة. وتكمن جدة هذا البحث في سدّ هذه الفجوة من خلال تطوير إطار عمل شامل لتقييم الهشاشة الزلزالية، مُصمم خصيصاً للمباني المبنية من الطوب غير المسلح في مدينة السليمانية بالعراق. تبدأ هذه الدراسة بمراجعة للتقنيات التجريبية والتحليلية المُعتمدة عالميًا لتقييم الهشاشة الزلزالية، مع توضيح نقاط قوتها وقيودها وتطبيقاتها النموذجية. في مدينة السليمانية، أظهر مسح ميداني شمل ٤٧٢٤ مبنى، استناداً إلى تصنيفEMS-98 ، أن معظم المباني تتدرج ضمن فئتي الهشاشة Bو Cفي ظل السيناريوهات المتشائمة والمتفائلة على التوالي، وأنها تُبنى في الغالب باستخدام كتل خرسانية مجوفة. تم تحليل مبنى نموذجي من الطوب غير المسلح باستخدام التحليل الاستاتيكي غير الخطي، وذلك بتطبيق طريقة الإطار المكافئ في برنامج TREMURI، مع استخدام خصائص المواد المستمدة من الاختبارات التجريبية (متوسط مقاومة الضغط ≈ ٣.٠ ميجا باسكال ومعامل المرونة ≈ ٢٤٠٠ ميجا باسكال). وكشفت النتائج عن انخفاض في المقاومة الجانبية والصلابة في الاتجاه السيني، مما يؤدي إلى زيادة احتمالية حدوث أضرار طفيفة عند مستويات منخفضة من تسارع ذروة الأرض) حوالي ٠.٠٥ (gمقارنةً بالاتجاه الصادي الأكثر مقاومة، مما يُبرز أهمية مراعاة السلوك الاتجاهي وخصائص المواد المحلية في الحد من مخاطر الزلازل.

الكلمات المفتاحية

الهشاشة الزلزالية، البناء غير المسلح، الأساليب التجريبية، الأساليب التحليلية، تحليل التاريخ الزمني، تحليل الدفع، منحنيات الهشاشة. مدينة السليمانية

Figure 1: Various scales of seismic vulnerability assessment approaches

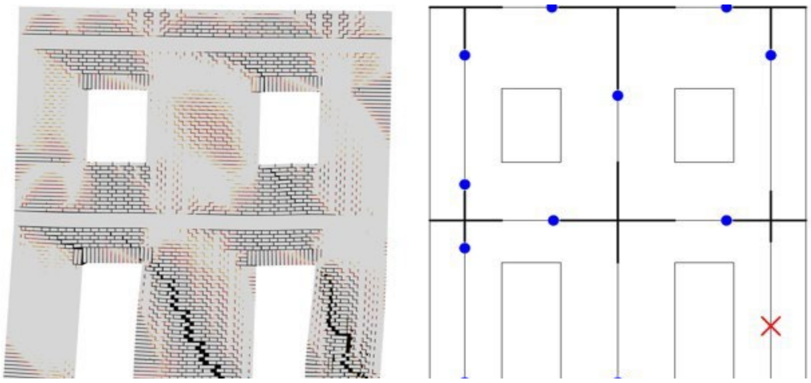


Figure 2: a) Micromodel b) Equivalent Frame Model

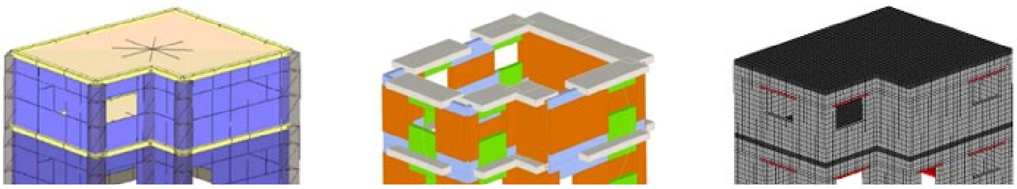


Figure 3: Numerical models developed using different approaches: (a) 3DMacro spring-based macro-element simulation, (b) REMURI modeling using beam-type macro-elements, (c) DIANA FEA continuum modeling approach

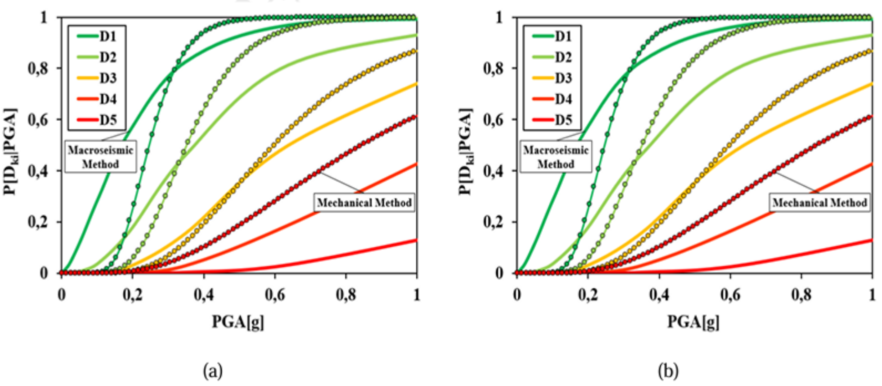


Figure 4: Comparison of fragility curves: (a) x-direction, (b) y-direction

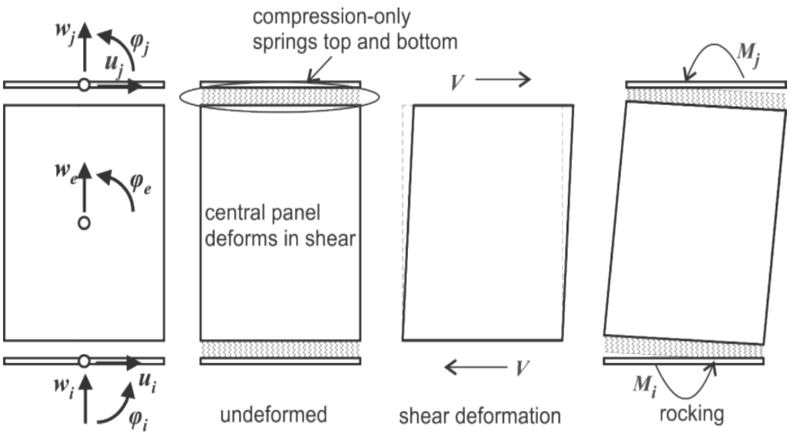


Figure 5: Kinematics of macro-element

Figure 6: a) Prism compression test   b) Diagonal compression test.

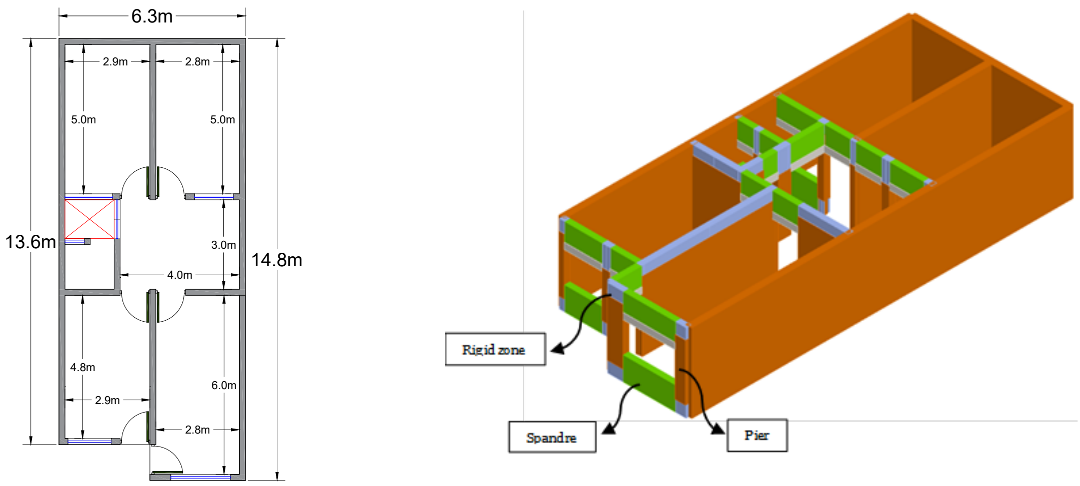


Figure 7: Model and distributed pier, spandrel and rigid nodes.

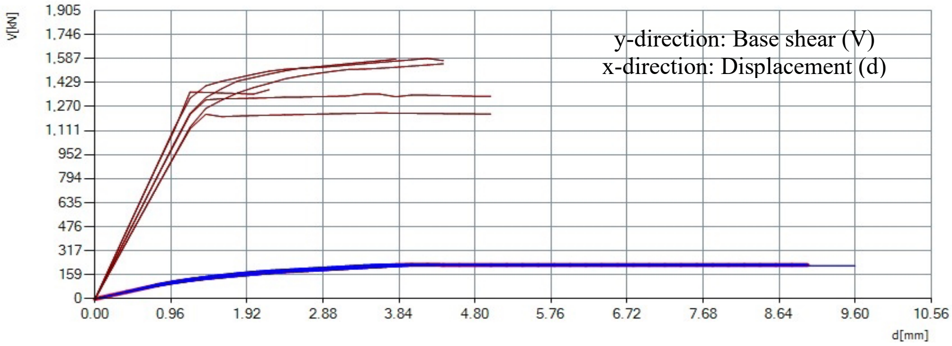


Figure 8: x-direction (blue) and y-direction (red) pushover curves.

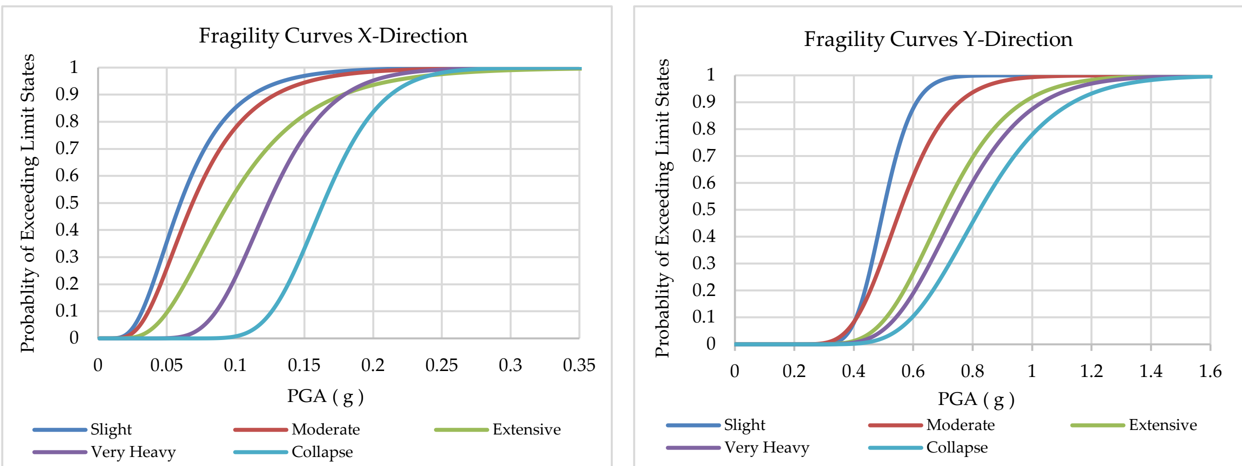
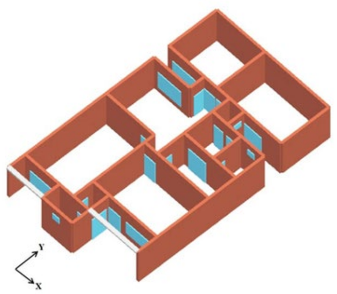
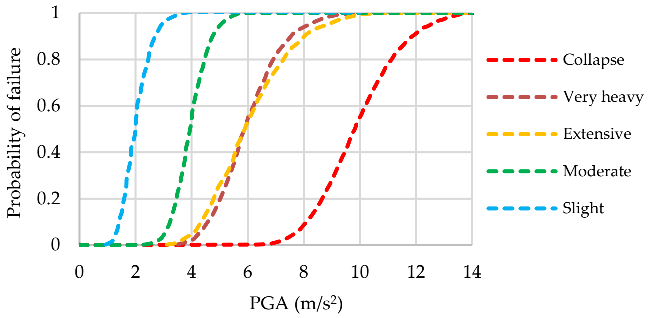


Figure 9: Fragility curve in x and y direction





a. b.

Figure 10: a) Three-dimensional view of the single-story buildings modeled in TREMURI. b) Fragility curves in x-direction

Table 1: Vulnerability categories assigned to different structures (EMS-98)

Construction Materials	Code	Lateral Force Resisting System	Range of Vulnerability Class	Most Likely Vulnerability Class
Masonry	M1	Rubble stone, fieldstone	A	A
	M2	Adobe (earth brick)	A–B	A
	M3	Simple Stone	A–B	B
	M4	Massive Stone	B–D	C
	M5	Unreinforced manufactured stone units	A–C	B
	M6	Unreinforced, with RC floors	B–D	C
	M7	Reinforced or confined	C–E	D
Reinforced Concrete (RC)	RC1	Frame without ERD*	A–D	C
	RC2	Frame with moderate level of ERD	B–E	D
	RC3	Frame with high level of ERD	C–F	E
	RC4	Walls without ERD	B–D	C
	RC5	Walls with moderate level of ERD	C–E	D
	RC6	Walls with high level of ERD	D–F	E
Steel	S	Steel structures	C–F	E
Wood	W	Timber structures	B–E	D

*Earthquake Resistant Design.

Table 2 Masonry material characteristics. (Researcher)

Material Characteristic	Value
Modulus of elasticity	2400 N/mm^2
Shear modulus	1000 N/mm^2
Specific weight	1500 kg/m^3
Mean compressive strength of masonry	3 N/mm^2
Shear strength	0.46 N/mm^2
Characteristic compressive strength of masonry	2.9 N/mm^2
Confidence factor	1
Partial safety factor for material	1.3
Shear drift	0.004
Bending drift	0.008
Final creep coefficient	0

Table 3: Parameters of the SDOF system for pushover in x- and y-direction

Parameter	Value (x-Direction)	Value (y-Direction)
T* (s)	0.177	0.059
m* (kg)	70,666	99,842
w (kN)	1,213	1,213
M (kg)	123,696	123,696
m*/M (%)	57.129	80.715
Γ	1.29	0.98
F^*_y (kN)	172	1,383
d^*_y (mm)	1.94	1.23
d^*_m (mm)	6.95	2.24

Table 4: Performance levels calculated for single- story URM buildings in KR based on Milutinovic and Trendafiloski (2003) criterion

Damage State	Limit Horizontal Displacement Equation	Displacement (mm) x-direction	Displacement (mm) y-direction
Slight	d=0.7dy	1.75	0.84
Moderate	d=0.7dy+0.05(0.9du−0.7dy)	2.06	0.90
Extensive	d=0.7dy+0.2(0.9du−0.7dy)	3.01	1.07
Very Heavy	d=0.7dy+0.5(0.9du−0.7dy)	4.91	1.41
Collapse	d=0.9du	8.06	1.97

List of Symbols

w	Axial displacements
u	Lateral displacements
φ	Rotations
d_y	Yield displacement
d_u	Ultimate displacement
P	Probability
Φ	Standard normal cumulative distribution function
x	Distributed intensity measure
D	Damage State
μ	Mean of the natural logarithm
β	Standard deviation of the natural logarithm