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1	Seismic fragility analysis of masonry structures considering the
2	effect of mainshock-aftershock sequences
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4	
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12	
13	Abstract: Earthquake investigations indicate that strong aftershocks can further
14	aggravate the structural damage and low attention has been given to understand the
15	seismic performance of masonry structures under seismic sequences. This manuscript
16	studies the seismic fragility of masonry structures considering the effect of aftershocks
17	by proposing a simplified probabilistic approach. In total, 36,000,000 stochastic
18	earthquake-structure samples were generated using Monte Carlo simulation to consider
19	the uncertainty of earthquake ground motions and structures, while seismic fragility
20	curves of masonry structures were obtained by considering the following parameters:

21 1) aftershock intensity, 2) seismic wall area ratio, 3) site conditions, 4) the number of 22 storeys, 5) reinforced concrete (RC) tie column, 6) mortar strength. The interstorey drift 23 ratio was employed to assess the structural performance levels. The results showed that 24 strong aftershocks have a significant influence on the fragility curves of masonry 25 structures. The probability of exceeding the collapse limit state of structures can 26 increase by 32.2% due to aftershock effect. Among the examined parameters it was 27 found that the number of storeys has the greatest effect on the seismic fragility of 28 masonry structures. By increasing the number of storeys, the probability of exceedance 29 of the collapse damage state can increase up to 28.7%. The derived fragility curves are 30 validated against a finite element method, which indicate the rationality of the proposed 31 methodology.

32 Key words: masonry structures; mainshock-aftershock sequences; fragility curves;
33 seismic performance assessment; uncertainty

34

35 **1 Introduction**

Past earthquakes showed that a major earthquake (mainshock) is always followed by secondary earthquakes (aftershock), and the mainshock and the aftershocks constitute a mainshock-aftershock (M-A) sequence. Although the magnitude of aftershock tends to be lower than that of the mainshock, the aftershock can still produce moderate and strong ground motions that can further aggravate the structural damage produced by the mainshock. The aftershock events of the 1999 Chi-Chi earthquake [1],

the 2008 Wenchuan earthquake [2], the 2010 New Zealand earthquake [3], the 2011 42 Tohoku earthquake [4], and the 2015 Nepal earthquake [5] further increased structural 43 44 damage of structures caused by the main event and in some cases led to the structural 45 collapse, resulting in an increase in casualties and economic losses. Meanwhile, it has 46 been observed that no effective repair or strengthening works can be done within short 47 interval between a mainshock and the corresponding aftershock [6]. To investigate the 48 effect of aftershocks, several researchers have studied the effect of M-A sequences on 49 structures, mainly on reinforced concrete frames [7][8][9][10], steel frames 50 [11][12][13], and wooden structures [14][15]. However, less attention has been paid to 51 masonry structures [16], and the related fields still need to be further studied.

52 Masonry structures have been widely used in China due to its simple construction 53 and low cost. However, masonry structures exhibit obvious brittle characteristics and 54 poor structural integrity due to the low tensile, flexural and shear strength of masonry. 55 As a result, masonry structures are seriously damaged by earthquakes [17], especially, 56 under M-A sequences. Vulnerability assessment is a commonly used method to evaluate 57 structural performance. Sun and Zhang [18] proposed seismic damage probability matrices based on 38 regions in 17 earthquake events. Asteris et al. [19] conducted a 58 59 vulnerability assessment method of historical masonry structures by utilizing a finite 60 element method. Borzi et al. [20] and Ahmad et al. [21] established a simplified 61 pushover-based method for masonry structures to obtain the fragility curves. 62 Lagomarsino et al. [22] proposed a macroseismic vulnerability model for existing

63 masonry buildings, which can be used for seismic risk assessment on a regional and national scale. Shabani et al. [23] reviewed the simplified analytical methods for the 64 65 seismic vulnerability assessment of unreinforced masonry structures. To consider the 66 effect of aftershocks, Rinaldin and Amadio [16] studied the structural response through 67 time history dynamic analysis on a single-degree-of-freedom (SDOF) system. It has 68 been seen from the above studies that although fragility analysis is a well-accepted method to analyze the seismic vulnerability of masonry structures, the effect of 69 70 aftershocks is ignored.

71 This manuscript tends to propose a simplified probabilistic approach for seismic 72 fragility of masonry structures. According to the location of RC tie columns of masonry 73 structures built in different times, the masonry structures are divided into five categories, 74 and the corresponding performance levels and interstorey drift ratio thresholds for assessing their seismic performance are defined. In total, 36,000,000 stochastic 75 76 earthquake-structure system models are developed to support the construction of the M-A fragility curves of masonry structures. The effects of the seismic wall area ratio, 77 78 aftershock intensity, site condition, number of storeys, RC tie column, and mortar 79 strength are systematically investigated.

80

2 Seismic vulnerability assessment

Fragility curves are often used to describe the conditional probability that a structure can reach or exceed for a given damage state resulted by subjecting the structure to a ground motion with a specific intensity. To obtain the seismic fragility curves of building structures, the peak ground acceleration (PGA) of ground motions is used as the intensity of input ground motions according to the current seismic design codes [24]. Considering the lognormal distribution assumption of structural response and damage limit state, the conditional probability $P(\cdot)$ that the interstorey drift ratio demand of structures $IDR_{max}|PGA$ exceeds the interstorey drift ratio capacity of damage limit state for a given PGA can be calculated from Eq.(1).

90
$$P(IDR_{\max} | PGA > IDR_{LSi}) = 1 - \Phi\left(\frac{\ln(IDR_{LSi}/\overline{IDR}_{\max})}{\sqrt{\beta_c^2 + \beta_d^2}}\right)$$
(1)

91 where *LSi* ($i = 1 \sim 5$) represents the *i*th damage limit state of structure; $\Phi(\cdot)$ is the normal 92 standard distribution function; *IDR*_{LSi} is the median interstorey drift ratio value of the 93 i^{th} damage limit state; $\overline{IDR}_{\text{max}}$ is the median value of the interstorey drift ratio response 94 of the structures; β_c represents the logarithmic standard deviation of the *IDR*_{LSi}; β_d 95 represents the logarithmic standard deviation of the *IDR*_{max}.

The calculation method of structural fragility curves for M-A sequences is the same as that of a single earthquake. The difference is that the structural demand for M-A sequences is larger, which is affected not only by the PGA of mainshock (PGA_{ms}), but also by the PGA of aftershock (PGA_{as}). Therefore, to calculate the fragility curves for M-A sequences, the *IDR*_{max}|PGA will be replaced by *IDR*_{max}|PGA_{seq} for M-A sequences in Eq.(1). The *IDR*_{max}|PGA_{seq} represents the interstorey drift ratio for M-A sequences.

103 To facilitate statistical analysis, the relative intensity of aftershock (γ) is introduced 104 and defined as the ratio of PGA_{as} to PGA_{ms}. To investigate the influence of aftershocks 105 with different intensity, 6 levels of γ is adopted: $\gamma = 0, 0.2, 0.4, 0.6, 0.8, \text{ and } 1.0$, where

106 $\gamma = 0$ considers only the mainshock effect.

107 **3** Structural capacity and seismic demand assessment

108

3.1 Structural performance levels

109 Performance-based seismic design has been established over the last decades to 110 set appropriate seismic performance objectives for structural design, so that the damage state and economic loss of the structure under severe earthquakes can meet the 111 112 requirements of the owner/stakeholder. The performance level describes the structural 113 damage limit state and the damage index required to define the damage limit state. 114 According to the Reference [24][25], five performance levels are considered: (a) Negligible (LS1), (b) Minor (LS2), (c) Moderate (LS3), (d) Severe (LS4), and (f) 115 116 Collapse (LS5), respectively. Common damage indices include the bearing capacity, 117 deformation capacity, and energy consumption capacity. Investigations show [26][27] 118 that storey displacement or interstorey drift ratio can comprehensively reflect the 119 damage state of masonry structures, which is widely applied in seismic design and 120 performance assessment.

121 There are great differences in the design of RC tie columns for masonry structures 122 in different areas in China due to economical reasons, therefore, masonry wall systems 123 are firstly classified to five classes according to the location of RC tie columns (see 124 Fig.1), namely as: class A (no RC tie columns), class B (RC tie columns located at blue 125 square in Fig.1), class C (RC tie columns located at blue and purple square in Fig.1),







141 Fig. 1. Plane of basic structure model and the location of RC tie columns (unit: mm).

142Table 1 Interstorey drift ratio limits for each performance level.

Performance level	Limit state	А	В	С	D	Е
Negligible	LS1			0.04%		

Minor	LS2			0.08%		
Moderate	LS3			0.13%		
Severe	LS4	0.26%	0.28%	0.31%	0.39%	0.46%
Collapse	LS5	0.39%	0.43%	0.52%	0.65%	0.79%

143 **3.2 Interstorey drift ratio demand**

In this study, the failure mode of multi-storey masonry structures is assumed to be dominated by shear, and the storey shear capacity-storey displacement curve is assumed to follow the principles of the elastic-perfectly plastic (EPP) model. The storey yield strength coefficient ξ_i , can be calculated as Eq. (2) [29], is defined as the ratio of the ultimate shear strength of the *i*th storey to the seismic shear force V_i of the *i*th storey.

149
$$\xi_{i} = \frac{0.9R_{ui}}{V_{i}} = \frac{0.11\rho_{i}}{\alpha_{\max}g_{e}} \cdot \frac{n+1}{(n+i)(n-i+1)} \sqrt{f_{2,i} + \frac{8.33g_{e}(n-i+1)\sqrt{f_{2,i}}}{\rho_{i} + \rho_{i}'}} \qquad (2)$$

where ξ_i is the storey yield strength coefficient of the *i*th storey; R_{ui} is the ultimate shear 150 capacity of i^{th} storey; V_i is the seismic shear force of the i^{th} storey; g_e is a combined 151 gravity load that consider the effect of dead load and live Load (1.0 Dead load + 0.5152 153 Live Load) [24]; *n* is the number of storeys; α_{max} is the maximum seismic influence 154 coefficient, which can be determined as $\alpha_{max}=2.25 \cdot PGA_{ms} \cdot (1+0.03 \cdot \gamma)$ for M-A sequences; ρ_i and ρ'_i is the seismic wall area ratio in calculation direction and 155 orthogonal direction of the i^{th} storey, which represents the ratio of the cross-sectional 156 area of masonry wall in half-storey height to the storey area; $f_{2,i}$ is average compressive 157 strength of the mortar used in i^{th} storey. The coefficient of 0.11 in Eq. (2) should be 158 159 changed to $0.11 \times 0.85 = 0.094$ [29] when *n* is equal to 1.

160 The strength reduction factor *R* can be expressed as the inverse of minimum ξ_i

161 (that is $\xi_{i,\min}$), which is shown in Eq. (3). Because RC tie column has a great influence 162 on storey yield strength coefficient ξ_i (described in Fig. 2), influence coefficient η is 163 introduced to consider the influence of RC tie columns on the lateral capacity of 164 masonry structures. According to *Standard for seismic appraisal of buildings* [30], for 165 class A, class B, class C, class D, and class E, the η are 1.0, 1.05, 1.1, 1.2, and 1.3 166 respectively.

167
$$R = \frac{1}{\eta \cdot \xi_{i,\min}} = \frac{\alpha_{\max} g_e n}{0.11 \rho_i \sqrt{f_{2,i} + \frac{8.33 n g_e \sqrt{f_{2,i}}}{\rho_i + \rho'_i}}}$$
(3)

168 Generally, the soft-storey of a regular masonry structure is located at the bottom
169 storey of buildings, so the *i* in Eq.(3) can be approximated to be 1.

170 Considering the effect of shear deformation, bending deformation and coupling of 171 wall limbs, Jiang et al. [31] proposed a calculation formula for the natural period $T_{0,e}$ of 172 multi-storey masonry structures, which is expressed by Eq. (4).

173
$$T_{0,e} = \left(0.132 + 0.050 \frac{H}{B}\right) \sqrt{\frac{g_e}{f_m^{1.5} h\rho}} \cdot H$$
(4)

174 where *H* and *B* are the structural height and width, respectively; f_m is the average 175 compressive strength of masonry; *h* is the height of the soft-storey.

176 The ductility factor μ is obtained by *R* through *R*- μ -*T* relationship [32]. In this 177 manuscript, the *R*- μ -*T* relationship proposed by Zhang [33], which is calculated as 178 shown in Eq. (5), is adopted, because the expression considers the effect of M-A 179 sequences.

180
$$R = 1 + \frac{a_0 \left(a_1 T_{0,e} + T_{0,e}^2 \right) \left(a_4 + \mu \right)}{\left(1 + a_2 T_{0,e} + a_3 T_{0,e}^2 \right) \left(1 + a_5 \mu \right)} \frac{1}{0.87 + 0.08e^{1.2\gamma}}$$
(5)

181 where a_0 , a_1 , a_2 , a_3 , a_4 and a_5 are parameters depending on site classes. The site classes

182 are determined according to
$$V_{20}$$
 [24] and listed in Table 2.

183

Table 2 The value of $a_0 \sim a_5$.

Site class	V ₂₀	V ₃₀	a_0	a_1	a_2	<i>a</i> ₃	<i>a</i> 4	<i>a</i> 5
Ι	$V_{20} > 500 \text{m/s}$	V ₃₀ > 596m/s	0.86	10.83	9.68	0.57	-0.79	0.02
II	$250 \text{m/s} < V_{20} \le 500 \text{m/s}$	$278 \text{m/s} < V_{30} \le 596 \text{m/s}$	0.71	13.21	9.97	0.98	-0.84	0.01
III	$150 \text{m/s} < V_{20} \le 250 \text{m/s}$	$158 \text{m/s} < V_{30} \le 278 \text{m/s}$	1.03	10.93	11.49	0.77	-0.95	0.04
IV	$V_{20} \le 150 \text{m/s}$	V ₃₀ ≤158m/s	0.66	13.25	9.95	0.55	-0.81	0.01

To evaluate the displacement response of masonry structures, an equivalent elastic single-degree-of-freedom system was proposed by Lin and Lin [34] is adopted, and the equivalent elastic period T_{eq} and the equivalent damping ζ_{eq} are computed by Eq. (6) and Eq. (7), respectively.

188

$$T_{eq} = T_{0,e} \sqrt{\mu} \tag{6}$$

(7)

189
$$\zeta_{eq} = \zeta_0 + 0.079 T_{0,e}^{-0.252} \sqrt{R - 1}$$

190 where ζ_0 is the elastic viscous damping coefficient.

191 Based on T_{eq} and ζ_{eq} , the damping reduction factor *B* can be determined by Eq. (8) 192 [29].

193
$$B = \begin{cases} 1 + \frac{0.05 - \zeta_{eq}}{0.06 + 1.4\zeta_{eq}} & (T_{eq} \le T_g) \\ \left(1 + \frac{0.05 - \zeta_{eq}}{0.06 + 1.4\zeta_{eq}}\right) \cdot \left(\frac{T_g}{T_{eq}}\right)^{0.9 + \frac{0.05 - \zeta_{eq}}{0.5 + 5\zeta_{eq}}} & (T_{eq} > T_g) \end{cases}$$
(8)

194 where T_g is the characteristic period, which can be obtained from Reference [24].

195 The yield spectral displacement S_{dy} and inelastic spectral displacement S_{dp} can be 196 obtained by Eq.(9) and Eq.(10) [35], respectively.

197
$$S_{dy} = \frac{S_{de}}{R} = \frac{T_{0,e}^2}{4\pi^2} \cdot \frac{\alpha_{\max}g}{R}$$
(9)

198
$$S_{dp} = \frac{T_{eq}^2}{4\pi^2} \cdot \alpha_{\max} g \cdot B$$
(10)

199 where S_{de} is the elastic spectral displacement of the single-degree-of-freedom (SDOF) 200 system.

201 The inelastic displacement δ_p of the soft storey can be obtained by Eq.(11) [36].

202
$$\delta_{p} = \delta_{y} + \frac{S_{dp} - S_{dy}}{0.8 + 0.1n} = \frac{h}{\Gamma_{h}H}S_{dy} + \frac{S_{dp} - S_{dy}}{0.8 + 0.1n}$$
(11)

203 where δ_v is the yield displacement of the soft storey; Γ_h is the modal height coefficient.

204 The maximum interstorey drift ratio IDR_{max} can be estimated as Eq. (12).

 $205 \qquad IDR_{\max} = \delta_p / h \tag{12}$

Base on the calculation of IDR_{max} , the flow charts for the seismic fragility function of masonry structures is presented in Fig.2.

The proposed method (Eq. (2)~Eq. (11)) applies to the masonry structures with a height up to 21m, dominated by shear deformation, and with a uniform distribution of mass and stiffness along the height of the building. This method is not applicable to irregular structures with significant torsional effects. Moreover, the equivalent base shear method, which assumed that the horizontal seismic load is distributed in an inverted triangle along the height of the building, was adopted to establish Eq. (2) and Eq. (3). The out-of-plane failure and the interaction between the in-plane and out-of-plane actions [37] is not considered in this study. When the vertical load of the masonry wall is small and the out-of-plane constraint is poor (e.g., wooden floor, large walls without out-of-plane support), the influence of out-of-plane damage should be considered, and further research is needed.



220

Fig. 2. Calculation flowchart of seismic fragility analysis of masonry structures.

222 The effect of hysteretic energy on structural damage is not considered in this

used as the damage index, the R- μ -T relationship of Eq. (5) will be replaced by the R- μ -D-T relationship [38][39], in which the D and μ are unknown and the calculation flowchart (see Fig. 2) cannot continue.

method. The reason is that when the two-parameter model (e.g. Park-Ang model) is

227 **4 Earthquake-structure system**

228 **4.1 Basic structure model (BSM)**

223

A 5-storey masonry structure was designed as the basic structure model (BSM), with a storey height of 3.0m, as shown in Fig. 1. The width and length of BSM is 9.3m and 39.6m, respectively. The thickness of all masonry walls is 240mm, so the ρ (Xdirection) and ρ ' (Y-direction) are 0.049 and 0.084, respectively. The g_e is 12.0kN/m². The compressive strength of mortar and masonry are 2.5MPa and 2.90MPa, respectively. The site condition of BSM is site class II.

4.2 Random variables to describe uncertainty in earthquake-structure system

236 The structural response is determined by the characteristics of structures and 237 earthquakes. As a result, the uncertainty of an earthquake-structure system includes 238 structural uncertainty and earthquake uncertainty. There are many factors causing the 239 uncertainty of masonry structures, including the uncertainty of materials and the uncertainty of geometry dimensions. The compressive strength of mortar f_2 and 240 241 masonry f_m are used as random variables to reflect the uncertainty induced by the 242 construction of masonry walls. The length L, width B, height h, and wall thickness t of 243 masonry structures are used as random variables to reflect the uncertainty induced by

244	the geometrical dimensions of the structures themselves. Moreover, to comprehensively
245	reflect the uncertainty of structures, it is also necessary to investigate the elastic
246	damping ratio ξ , the seismic wall area ratio ρ , and the combined gravity load g_e .
247	Moreover, the PGA _{ms} and characteristic period T_g are employed to reflect the
248	uncertainty induced to the analysis system by the M-A sequences. The lower value of
249	PGA _{ms} is 0. The curve of α_{max} , which is related to PGA of the input seismic motions, is
250	plotted in Fig. 3 according to Chinese code GB 50011-2010 [24]. The characteristics of
251	the parameter uncertainty of the BSM are shown in Table 3.

Table 3 Random variables used in basic structure model and earthquake motions.

	Mean Coefficient of		Distribution	0
Parameter	Value	variation	model	references
Height $h(m)$	3.0	5.0%	Normal	[20], [21]
Length $L(m)$	39.6	5.0%	Normal	[20], [21]
Width $B(m)$	9.3	5.0%	Normal	[20], [21]
Wall thickness <i>t</i> (m)	0.24	5.0%	Normal	[20], [21]
Compression strength of	• • • •	15.00/		[20], [21],
masonry f_m (MPa)	2.90	17.0%	Normal	[40]
Compression strength mortar				[20], [21],
f2(MPa)	2.5	30.0%	Normal	[40]
Seismic wall area ratio of Y-	0.004	2.5%	NT 1	[20]
direction wall ρ '	0.084	3.3%	Normal	[20]





253

Fig. 3. Seismic influence coefficient curve specific in GB 50011-2010 [24].

4.3 Earthquake-structure samples

Monte Carlo simulations were employed to generate random variables of earthquake-structure system. To analyze the effect of different parameters on the fragility curves of masonry structures, a total of 3,600 structure models were developed, corresponding to the earthquake-structure system with 2 levels of seismic wall area ratio, 6 levels of aftershock intensity, 4 levels of site condition, 3 levels of number of storeys, 5 levels of the location of RC tie columns, and 5 levels of mortar strength. Each model generates 10,000 earthquake-structure samples, thus 36,000,000 earthquakestructure samples were obtained by developing 3,600 structure models.

264 **5 Seismic fragility curves**

damage limit states.

269

270

According to the procedure described in Section 3, a total of 36,000,000 earthquake-structure samples are used to develop the seismic fragility curves. The fragility curves of masonry structures with different ρ is shown in Fig. 4, the X-axis represents PGA_{ms}, and the Y-axis represents the exceeding probability for different



271 (a) Seismic wall area ratio $\rho = 0.049$ (b) Seismic wall area ratio $\rho = 0.068$

Fig. 4. The fragility curves for M-A sequences with different ρ and γ .

For the same damage limit state, the exceeding probability of masonry structures increases as PGA_{ms} increases. Namely, with the increment of the earthquake intensity, more masonry structures exceed the given damage limit state. Moreover, under same PGA_{ms}, as the damage state worsens, the exceeding probability of the structures gradually decreases.



280	is 0.068. As shown in Fig. 4, when PGA _{ms} =0.2g and γ =1.0, the exceeding probability
281	of LS1 for ρ =0.049 and 0.068 are 86.6% and 79.1%, respectively. The exceeding
282	probability of LS3 for ρ =0.049 and 0.068 are 66.0% and 53.4%, respectively. The
283	exceeding probability of LS5 for ρ =0.049 and 0.068 are 28.7% and 20.2%, respectively.
284	The results show that the greater the ρ , the lower the vulnerability of masonry structures.
285	The earthquake load is mainly beared by the walls along the direction of earthquake
286	load. Therefore, the greater the ρ , the better the seismic performance, and the lower the
287	structural vulnerability.
288	When PGA _{ms} =0.2g and γ =0, 0.2, 0.4, 0.6, 0.8, and 1.0, the exceeding probability
289	of LS1 for BSM (see Fig. 4a) are 85.2%, 85.3%, 85.6%, 85.9%, 86.2%, and 86.6%,
290	respectively. The exceeding probability of LS3 for BSM are 60.0%, 61.4%, 62.1%,
291	63.8%, 64.7%, and 66.0%, respectively, while the exceeding probability of LS5 for
292	BSM are 21.4%, 22.6%, 24.4%, 25.6%, 27.2%, and 28.7%, respectively. The results
293	show that with the increment of the γ , M-A sequences induce higher levels of damage
294	resulting in greater vulnerability of the system. When γ is less than 0.6, the difference
295	of the exceeding probability between the mainshock analysis and M-A sequences
296	analysis was found to be less than 10%, indicating that the aftershock effect can be
297	ignored. When γ is greater than 0.6, the difference of the exceeding probability between
298	the mainshock analysis and M-A sequences analysis was found to be greater than 10%,
299	indicating that the aftershock effect should be considered in the preliminary design of
300	the structures. The exceeding probability of LS5 for an aftershock with γ =1.0 is 32.2%

301 higher (in average) than that when the structure is subjected to a mainshock only.

302 5.1 Effect of site condition

To investigate the effect of site condition on fragility curves, four site condition of BSM are considered: site class I, site class II, site class III, and site class IV, respectively, remaining the other parameters of BSM unchanged. Fig. 5 shows the fragility curves of the BSM with different site condition.

307 For the masonry structures located at site class I, site class II, site class III, and site class IV, when PGA_{ms}=0.2g and γ =1.0, the exceeding probability of LS1 are 74.5%, 308 309 86.6%, 92.9%, and 89.2%, respectively; the exceeding probability of LS3 are 49.8%, 310 66.0%, 73.0%, and 69.9%, respectively; the exceeding probability of LS5 are 15.3%, 28.7%, 39.6%, and 33.7%, respectively. The results show that there are significant 311 312 differences in the seismic fragility curves of masonry structures located at different site 313 condition. The reason is that the characteristic periods $T_{\rm g}$ of each site class are different, 314 among which the characteristic period of class I site is 0.2-0.35s, site class II is 0.35-315 0.45s, site class III is 0.45-0.65s, and class IV site is 0.65-0.90s. However, the initial period T₀ of the BSM is approximately 0.39s. According to Code for seismic design of 316 buildings [24], the spectral acceleration of site class I is significantly smaller than that 317 318 of other site classes, resulting in smaller vulnerability for the system. Due to the randomness of T_0 and T_g , the T_g of site class II might also be smaller than the T_0 . As a 319 320 result, the vulnerability of the system for site class II is smaller than that of site class 321 III. For site class III and site class IV, the latter has a greater strength reduction factor

than the former, so the effect of the latter is smaller than the former, thus system'svulnerability of the latter is smaller than the former.

For the masonry structures located at site class I, site class II, site class III, and site class IV, the exceeding probability of LS1 for γ =1.0 is 1.04, 1.01, 1.01, and 1.02 times of that for γ =0, respectively, while the exceeding probability of LS5 for γ =1.0 is 1.36, 1.23, 1.19, and 1.21 times of that for γ =0, respectively. The results show that the increase of the exceeding probability for site class II, III and IV is basically the same under the same γ , while the increase of the exceeding probability for site I was found to be larger, indicating that aftershocks have greater impact on masonry structures located





Fig. 5. The fragility curves of the masonry structures in different site classes.

337 5.2 Effect of number of storeys

To study the effect of the number of storeys n, the n is changed based on the BSM, and the n is set as 3, 4, and 5, respectively, the fragility curves for different n are are shown in Fig. 6.

341 As shown in Fig. 6, *n* has a significant impact on the seismic fragility curves of 342 masonry structure. When PGA_{ms} is 0.2g and γ =1.0, the exceeding probability of LS1 for masonry structures with n = 3, 4, and 5 are 59.2%, 78.0%, and 86.6%, respectively. 343 344 The exceeding probability of LS3 for structures with n = 3, 4, and 5 are 28.3%, 50.7%, 345 and 66.0%, respectively. Accordingly, the exceeding probability of LS5 for structures with n = 3, 4, and 5 are 6.7%, 18.2%, and 28.7%, respectively. The results show that 346 the exceeding probability increases significantly as *n* increases. Namely, the larger the 347 348 n, the more severe the structural damage. As the n or total height increase, the 349 overturning moment of the masonry structure increases, resulting in more severe 350 damage to the structures induced by the greater base shear.

Earthquake damage investigation showed that the degree of structural damage in the same intensity zone is proportional to *n*. The larger the *n*, the higher the percentage of damage or collapse.

For masonry structures with n = 3, 4, and 5, the exceeding probability of LS1 for $\gamma=1.0$ is 1.04, 1.03, and 1.02 times of that for $\gamma=0$, while the exceeding probability of LS5 for $\gamma=1.0$ is 1.42, 1.33, and 1.24 times of that for $\gamma=0$. The results show that the effect of aftershocks increases with the decrease of the number of storeys.





363 **5.3 Effect of RC tie column location**

To investigate the effect of the location of RC tie columns, based on the BSM, five locations of RC tie columns, namely class A, class B, class C, class D and class E (see Fig.1), are adopted to calculate the fragility curves, as shown in Fig. 7.

For PGA_{ms} = 0.2g and γ =1.0, the exceeding probability of LS1 for class A, class B, class C, class D and class E are 86.6%, 85.6%, 84.0%, 81.8%, and 80.3%, respectively; the exceeding probability of LS3 for class A, class B, class C, class D and class E are 66.0%, 63.2%, 59.8%, 54.5%, and 49.1%, respectively. The difference of exceeding probability of LS1 for masonry structures with different location of RC tie columns is very small, which is also the same for the exceeding probability of LS2 and

373	LS3. The phenomenon indicates that the RC tie column has little influence on the
374	structural performance before yielding. For PGA _{ms} =0.2g and γ =1.0, the exceeding
375	probability of LS5 are 28.7%, 23.5%, 15.8%, 8.9%, and 4.9%, respectively. The
376	exceeding probability of LS5 for masonry structures with different location of RC tie
377	columns have a significant difference. The results show that reasonable setting of RC
378	tie columns can reduce the exceeding probability of LS5 for 5-storey masonry structure
379	from 28.7% to 4.9%, indicating that the RC tie column has a significant influence on
380	the structural performance after yielding. The main reason is that RC tie column is
381	effective at larger displacements to confine the masonry, thus improving the structural
382	integrity and structural deformation capacity. Before yielding, the structural integrity is
383	good, and the influence of RC tie column is small. After yielding, the RC tie column
384	effectively ensures the structural integrity and improves the structural vulnerability, and
385	the influence of RC tie column is significant. By increasing the number of RC tie
386	columns in a masonry structure, the restraint of RC tie columns on masonry walls is
387	stronger, the shear bearing capacity of the structure increases, displacements are
388	reduced, and less damage is expected.

When the same damage limit for the interstorey drift ratio is adopted for masonry structures with different locations of RC tie columns, the damage state of masonry structure with class E will be overestimated while that of class A will be underestimated. As a result, for a given damage limit state, masonry structures with different locations of RC tie columns should adopt different limit values for the interstorey drift ratio, as shown in Table 1.

395 For the masonry structures with class A, class B, class C, class D and class E, the 396 exceeding probability of LS1 for γ =1.0 is 1.01, 1.02, 1.02, 1.02, and 1.02 times of that 397 for $\gamma=0$, respectively, while the exceeding probability of LS5 for $\gamma=1.0$ is 1.23, 1.28, 398 1.34, 1.44, and 1.54 times of that for $\gamma=0$, respectively. The results show that aftershocks 399 have greater impact on the exceeding probability of LS4 and LS5 (see Fig. 7), indicating that aftershocks have a great influence on the masonry structure after yielding, but have 400 401 small influence on the elastic region of the structural response. Moreover, the effect of 402 aftershocks increases with the increase of the number of RC tie columns. $\gamma = 0.2$ =0.2=0.2



407 Fig. 7. The curves of masonry structures with different setting of RC tie columns.

408 **5.4 Effect of mortar strength**

To study the effect of mortar strength f_2 , based on the BSM, five mortar strengths, 409 namely the $f_2 = 1.0$ MPa, 2.5MPa, 5.0MPa, 7.5MPa and 10.0MPa, are adopted to 410 411 calculate the seismic fragility curves, as shown in Fig. 8. 412 When PGA_{ms}=0.2g and γ =1.0, the exceeding probability of LS1 for masonry 413 structures with *f*₂ = 1.0MPa, 2.5MPa, 5.0MPa, 7.5MPa and 10.0MPa are 94.7%, 86.6%, 75.3%, 64.2%, and 53.9%, respectively. The exceeding probability of LS3 for masonry 414 415 structures with *f*₂ = 1.0MPa, 2.5MPa, 5.0MPa, 7.5MPa and 10.0MPa are 82.9%, 66.0%, 416 47.6%, 34.8%, and 25.3%, respectively. The exceeding probability of LS5 for masonry 417 structures with *f*₂ = 1.0MPa, 2.5MPa, 5.0MPa, 7.5MPa and 10.0MPa are 44.8%, 28.7%, 17.2%, 11.0%, and 7.2%, respectively. The results show that as the f_2 increases, the 418 419 structural performance increases significantly, and the vulnerability of the system gradually decreases. The shear strength of masonry increases with the increase of f_2 , 420 421 and the seismic capacity of the structures is improved. As a result, the stronger the 422 mortar strength the lower the probability of exceeding a certain damage state. 423 For the masonry structures with $f_2 = 1.0$ MPa, 2.5MPa, 5.0MPa, 7.5MPa and 10.0MPa, the exceeding probability of LS1 for γ =1.0 is 1.01, 1.01, 1.03, 1.04, and 1.05 424 425 times of that for $\gamma=0$, respectively, while the exceeding probability of LS5 for $\gamma=1.0$ is 426 1.19, 1.24, 1.31, 1.38, and 1.47 times of that for $\gamma=0$. The results show that the effect of

427 the aftershocks increases with the increase of the f_2 , respectively.



432 Fig. 8. The fragility curves of masonry structures with different f_2 .

433 **5.5 Validation**

434 5.5.1 Comparison between shake table tests and the proposed method

435 A part of the BSM was modeled (the shaded part of the masonry structure shown 436 in Fig. 1) to verify the proposed approach for *IDR* demand of masonry structures. The 437 shake table tests [41] were adopted to obtain IDR of the 1/4-scaled model, and the 438 calculated results are compared with those calculated by the proposed method, as shown 439 in Table 4. The measured natural period of the 1/4-scaled model is 0.126s and the 440 similarity coefficient is 3.162:1, so the natural period of the BSM is 0.397s. The calculated natural period is 0.412s, and the error of the natural period is 3.8%, indicating 441 that the calculated and experimental results have good consistency for the dynamic 442

443 properties of the building structure. When the input $PGA_{ms} = 0.035g$, 0.10g, and 0.22g 444 for mainshock only, the errors of IDR_{max} between calculated and average experimental 445 results are all within 10.0%, indicating that the proposed method is reasonable.

446

Table 4 Validation of IDR_{max} of masonry structures.

	Experimental results		Numerica	al results	
	El Centro	Taft	El Centro	Taft	- Calculated results
0.035g	1/2941	1/3393	1/2959	1/3217	1/3030
0.10g	1/1167	1/789	1/1214	1/744	1/857
0.22g	1/142	1/158	1/153	1/171	1/153

447 5.5.2 Comparison with fragility curves obtained from finite element method

Considering that fragility curves under M-A sequences are not available [42], the 448 449 more accurate macro-modeling finite element method (FEM) which has been 450 previously validated by the authors using shake table tests is adopted in this section to 451 obtain the fragility curves of the 5-storey BSM (the shaded part of the masonry structure 452 shown in Fig. 1). The natural periods of experimental and FEM results are 0.397s and 453 0.405s, and the error is 2.0%. It can be seen from Table 4 that the errors of *IDR* between 454 experimental and FEM results are all within 10%. Then, the numerical results are 455 compared with those obtained from the proposed calculation method. In macro-456 modeling finite element method, multilayer shell elements are used to simulate the 457 masonry walls and reinforced concrete floors, as shown in Fig. 9. The macro-modeling 458 method simplifies the brick and mortar into a homogenous material, the mechanical 459 properties of which have been determined by both the brick and mortar. The plasticity model is employed to consider the constitutive laws of masonry and concrete, while the 460 kinematic hardening model is used as the constitutive law of steel. The measured 461 462 average compressive strength of masonry and mortar are 3.3MPa and 1.5MPa, respectively. The specific value of other material strengths can be referred to [41]. The 463 464 failure of the element is determined by the maximum strain and stress rather than 465 cumulative damage. To consider the uncertainty of ground motions, 8 M-A sequences, which are recorded from site class II, are selected from 8 earthquake events and used 466 467 for the time history dynamic analysis. The specific finite element model and M-A sequence records can be found elsewhere [29]. Then, the interstorey drift ratios of the 468 5-storey BSM are calculated using dynamic time history analysis. Finally, the seismic 469 470 fragility curves are obtained by adopting the incremental dynamic analysis method. The fragility curves of the BSM calculated by the above two methods subjected to M-A 471 472 sequences with $\gamma=0$ and $\gamma=1.0$ are shown in Fig. 10. As expected, a good agreement can 473 be observed between the seismic fragility curves obtained by the finite element method 474 and the proposed method, which preliminary shows the feasibility of the proposed method. It should be noted that the dynamic time history analysis method takes nearly 475 476 a week to completion, while the proposed method takes only 2 hours.





Fig. 9. The finite element model. 478







481 Fig. 10. Comparison between the fragility curves calculated by the finite element 482 method and the proposed method in this manuscript.

5.6 Recommendation and Discussions 483

484 According to the analysis of the obtained fragility curves, the number of storeys 485 has the greatest effect on the fragility of masonry structures, followed by the location 486 and number of RC tie columns. The seismic wall area ratio and mortar strength follow, 487 while it was found that the site conditions have the lowest impact. Since the geometric 488 dimensions and the number of storeys has been pre-defined by the architects before structural analysis, the most effective way to improve the seismic performance of 489

490 masonry structures is by placing a sufficient number of RC tie columns in original 491 design. Using mortars with high strength is an alternative way to improve seismic 492 performance. The effect of aftershock should be considered in the preliminary design 493 of the structures when γ is less than 0.6.

494 **6** Conclusion

This manuscript focuses on a simplified probabilistic approach for seismic fragility analysis of masonry structures considering the influence of aftershocks. 36,000,000 stochastic earthquake-structure system samples were generated by Monte Carlo simulation to calculate seismic fragility curves. The effect of aftershock intensity, seismic wall area ratio, site condition, number of storeys, RC tie column, and mortar strength were studied. The following conclusions can be drawn:

(1) Based on the characteristics of the soft storey mechanism of masonry structures, a simplified probabilistic approach for seismic fragility analysis of masonry structures under M-A sequences is proposed. The uncertainty of masonry structures and the uncertainty of earthquake ground motions are considered in the proposed method to generate a large database of earthquake-structure samples. Compared with the finite element methods, the proposed method can save computational time significantly while maintaining accuracy.

508 (2) Strong aftershocks can further aggravate the damage state of masonry 509 structures. As the relative intensity γ of aftershock increases, structural vulnerability 510 increases gradually. The effect of aftershock on structural performance can be ignored

511	when γ is less than 0.6. The probability of exceeding the collapse limit state of structures
512	can increase by 32.2% when γ is equal to 1.0 (i.e., aftershock having equal intensity
513	with the mainshock).
514	(3) The number of storeys n has the greatest influence on the seismic vulnerability
515	of masonry structures. It was found that the exceeding probability of collapse damage
516	state increases from 6.7% to 28.7% with the increases of <i>n</i> .
517	(4) RC tie columns can enhance the seismic performance of masonry structures,
518	especially after yielding. By increasing the number of RC tie columns and selecting
519	properly their location across the floor plan of the structure, the exceeding probability
520	of collapse damage state decreases from 28.7% to 4.9%.
521	(5) The seismic wall area ratio ρ and the mortar strength f_2 strongly affects the
522	seismic vulnerability of masonry structures. With the ρ increasing from 0.049 to 0.068,
523	the exceeding probability of collapse damage state decreases from 28.7% to 20.2%.
524	With the f_2 increasing from 1.0MPa to 10.0MPa, the exceeding probability of collapse

525 damage state decreases from 44.8% to 7.2%.

526

527 CRediT authorship contribution statement

Yongqun Zhang: Conceptualization, Methodology, Investigation, Writing original draft, Funding acquisition. Zhuolin Wang: Investigation, Formal analysis,
Writing - original draft. Lixue Jiang: Supervision, Writing - review & editing.
Konstantinos Skalomenos: Supervision, Writing - review & editing. Dongbo Zhang:

532 Validation, Writing - original draft.

533

534 Declaration of Competing Interest

535 None.

536

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