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# 1 Seismic analysis method of unreinforced

# 2 masonry structures subjected to mainshock-

# 3 aftershock sequences

# 4 Yongqun Zhang<sup>1</sup>, Zhuolin Wang<sup>1</sup>, Lixue Jiang<sup>1\*</sup>, Konstantinos Skalomenos<sup>2</sup>, 5 and Dongbo Zhang<sup>1</sup>

### 6 Abstract

7 Aftershocks have the potential to further aggravate the damage of masonry structures caused 8 by mainshock. To quantitatively analyze the effect of aftershocks, this paper investigates the 9 seismic response of unreinforced masonry structures subjected to mainshock-aftershock (M-10 A) sequences. Firstly, an analytical method for estimating the maximum storey drift of masonry structures subjected to M-A sequences is proposed, which is based on the non-iterative 11 equivalent linearization method and the soft-storey failure mechanism of multi-storey masonry 12 13 structures. Then, a finite element method is employed to verify the effectiveness of the proposed method. Finally, a parametric analysis is performed to evaluate the effects of 14 15 aftershock intensity, anti-seismic wall area ratio, site classes, number of storeys, and mortar 16 strength on the seismic responses of masonry structures subjected to M-A sequences, 17 respectively. The results indicate that an excellent agreement for the maximum storey drift  $(\theta_{\max})$  between analytical and numerical results. The effect of aftershocks on masonry 18 19 structures in plastic phase is more distinct than that in elastic phase. Furthermore, the effect of 20 aftershocks on the  $\theta_{max}$  of masonry structures can be ignored when the relative intensity of 21 aftershock is less than 0.5, and the  $\theta_{max}$  can increase by approximately 19.0% when the relative 22 intensity of aftershock is equal to 1.0. Additionally, for the masonry structures subjected to M-A sequences, the effects of site classes on the  $\theta_{max}$  cannot be ignored, the  $\theta_{max}$  can decrease 23 24 with increasing anti-seismic wall area ratio and mortar strength, and increases with increasing 25 number of storeys.

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Keywords Masonry structure, Equivalent linear system, Storey yield strength
 coefficient, Seismic analysis, Aftershock

28

#### 29 **1. Introduction**

30 Earthquakes are not single events. It is common for a major earthquake (mainshock) to be 31 followed by many earthquakes (aftershocks) with lower magnitude which usually originated 32 at or near the rupture zone of the mainshock. Within 3 days after the Mw7.9 earthquake in 33 Wenchuan on May 12, 2008, approximately 3 aftershocks with magnitudes greater than 6.0 34 occurred (Wang et al. 2020). For the Mw8.8 Chile earthquake on February 27, 2010, about 90 35 aftershocks with magnitudes larger than 5.0 were recorded (USGS 2010). In the Mw7.8 Nepal 36 earthquake on April 25, 2015, 3 aftershocks with magnitudes larger than 6.0 were recorded 37 within 15 days after the mainshock (Apil et al. 2015). Generally, there is no enough time to 38 repair structures effectively due to the short interval between the mainshock and the aftershock 39 (Yeo et al. 2009). Therefore, aftershocks have the potential to further increase damages 40 significantly in the already damaged structures cuaused by the mainshock, resulting in 41 aggravation of economic losses and casualties. For the Turkey Van earthquake sequences, 42 28,000 buildings were damaged in the Mw7.1 mainshock on October 23, 2011, while 35,000 43 buildings were damaged after the Mw5.6 aftershock on November 9, 2011 (Ates et al. 2013). 44 For the New Zealand earthquake sequences, 100 people were injured in the Mw7.1 mainshock 45 on September 4, 2010, but 185 people were killed in the Mw6.3 aftershock on February 22, 2011 (Gledhill et al. 2011). Thus, the effect of aftershock should be considered for the seismic 46 47 performance assessment of building structures.

48 Several studies have focused on the effect of mainshock-aftershock (M-A) sequences on 49 the seismic response of building structures. Among them, the nonlinear dynamic analysis 50 method was usually employed to evaluate the structural behavior (e.g., displacement, storey 51 drift, damage index) under artificial or recorded M-A sequences (Hatzigeorgiou and Liolios 52 2010; Goda and Salami 2014; Shen et al. 2019; Zhang et al. 2020; Shen et al. 2020; Wang et 53 al. 2020). Noteworthily, the conclusions reached by different researchers diverged obviously. 54 Li and Ellingwood (Li and Ellingwood 2010) found that aftershocks had a significant effect on 55 structural damage, while Tesfamariam and Goda (2015) revealed that aftershocks had a 56 relatively minor effect. The reason is that factors such as site conditions and aftershock 57 intensity have a great influence on the results. To fully understand the effect of M-A sequences,

incremental dynamic analysis (IDA) and Monte Carlo simulation have been applied to attain
the structural vulnerability curve and study the effect of different aftershock intensities and
earthquake regions (Raghunandan et al. 2015; Li et al. 2020).

61 The main focus in seismic assessment of masonry structures has been on mainshock 62 analysis. Researchers carried out many quasi-static tests and shaking table tests to investigate 63 the failure pattern, bearing capacity, deformation capacity, and energy dissipation capacity of 64 masonry structures. A series of research results have provided a theoretical basis for the 65 performance assessment of masonry structures subjected to single earthquakes (Tomaževič 2007; Mendes and Loureno 2014; Graziotti et al. 2017; Guerrini et al. 2017; Nakamura et al. 66 67 2017; Azizi-Bondarabadi et al. 2019; Tomić et al. 2021). Rinaldin and Amadio (2018) 68 investigated the seismic behaviour of masonry structures under repeated earthquakes, a series 69 of non-linear dynamic analyses were employed to estimate the cumulative damage occurred 70 during the seismic sequence. The investigation has focused on the seismic response of masonry 71 structures subjected to M-A sequences, including displacement and damage index. However, 72 the peak ground acceleration (PGA) of the selected aftershocks in previous studies were equal 73 or similar to that of mainshock, which was impractical to quantitatively analyze the effect of 74 different aftershock intensities on the structural response. In addition, there is a lack of analysis 75 on the effect of masonry structure characteristics, such as material strength, number of storeys 76 and site conditions, to the structural response under M-A sequences.

77 This study aims to analyze the seismic response of masonry structures subjected to M-A 78 sequences. Firstly, a seismic analysis method for determining the maximum storey drift of 79 masonry structures is proposed based on the non-iterative equivalent linearization method and 80 the soft-storey failure mechanism of multi-storey masonry structures. Then, the effectiveness 81 of the proposed method is verified computationally using the finite element method on the 82 basis of shaking table tests. Finally, the effects of aftershock intensity, anti-seismic wall area 83 ratio, site classes, number of storeys, and mortar strength on the structural response are studied 84 systematically.

85

## 86 2. Calculated method of masonry structures subjected to M-A sequences

87 2.1 Storey yield strength coefficient of masonry structures

88 The equivalent base shear method is adopted for calculating the horizontal seismic load in 89 seismic analysis of masonry structures. It is assumed that the horizontal seismic load is 90 distributed in an inverted triangle along with the height of the structure. For a masonry 91 structure, the number of storeys is n, the building area of each storey is  $A_0$ , and the combined gravity load per unit building area is ge. According to the Code for seismic design of buildings 92 93 (GB50011-2010) (2010), the equivalent mass coefficient 0.85 is introduced to consider the 94 high mode effects of multi-storey masonry structures as in EC 8(2004). G<sub>eq</sub> is a combined 95 gravity load, which is defined as 1.0 Dead load + 0.5 Live Load (GB50011–2010).  $\alpha$  is the 96 seismic influence coefficient of sequence-type earthquake, which indicates the intensity of M-97 A sequences. Since the natural period of vibration of masonry structures is generally between 98 0.1s and 0.5s,  $\alpha$  is suggested to be equal to the maximum seismic influence coefficient  $\alpha_{max}$ 99 (GB50011–2010). Thus, the total base shear force  $V_0$  of the multi-storey masonry structure can 100 be calculated as:

$$V_0 = \alpha \cdot G_{eq} = 0.85 \alpha g_e n A_0 \tag{1}$$

102 The seismic shear force  $V_i$  of  $i^{th}$  storey can be estimated as

103 
$$V_i = 0.85 \alpha g_e A_0 \cdot \frac{(n+i)(n-i+1)}{(n+1)}$$
(2)

104 The plane and vertical layouts of the masonry structure of residential and office buildings 105 are generally regular, and these multi-storey masonry structures mostly adopt reinforced 106 concrete floors. The traditional method for calculating the vertical stress of the wall does not 107 consider the joint operation of the transverse and longitudinal walls. Therefore, the vertical 108 stresses of the transverse and longitudinal walls are quite different. Considering the fact that 109 the transverse and longitudinal walls work together, the vertical stresses of the connected 110 transverse and longitudinal walls tend to show a uniform distribution, and this trend is more 111 obvious in the lower storeys (Zheng and Jiang 2014). For the convenience of analysis, it is 112 assumed that the vertical stresses of the transverse and longitudinal walls in the same storey 113 are equal, and the floor is assumed to be rigid, thus the average ultimate shear capacity of  $i^{th}$ 114 storey can be estimated as

115

$$R_{ui} = A_{w,i} f_{vE,mi} = \rho_i A_0 f_{vE,mi}$$
(3)

116 where  $\rho_i$  is the anti-seismic wall area ratio in the calculation direction of the *i*<sup>th</sup> storey, which 117 can be expressed as the ratio of wall area  $A_{w,i}$  in half-storey hight to  $A_0$ ;  $\rho'_i$  is the anti-seismic 118 wall area ratio in the orthogonal direction of the *i*<sup>th</sup> storey. 119 The average seismic shear strength  $f_{vE,mi}$  of the  $i^{\text{th}}$  storey is calculated as (GB50011–2010)

$$f_{vE,mi} = \zeta_{Ni} f_{v,mi} \tag{4}$$

121 
$$f_{v,mi} = 0.125 \sqrt{f_{2,i}}$$
(5)

122 
$$\zeta_{Ni} = \frac{1}{1.20} \sqrt{1 + \sigma_i / f_{\nu,mi}}$$
(6)

where,  $f_{v,mi}$  is the average bond-slip strength of the masonry of the *i*<sup>th</sup> storey;  $f_{2,i}$  is the compressive strength of mortar of the *i*<sup>th</sup> storey;  $\zeta_{Ni}$  is the influence coefficient of vertical pressure of the *i*<sup>th</sup> storey;  $\sigma_i$  is the average vertical compressive stress in the *i*<sup>th</sup> storey due to gravity load, which be expressed as

127 
$$\sigma_i = \frac{g_e(n-i+1)}{\rho_i + \rho'_i} \tag{7}$$

128 where  $\rho'_i$  is the anti-seismic wall area ratio in the orthogonal direction of the *i*<sup>th</sup> storey.

129 Substitute Eq. (7) into Eq. (6), then  $\zeta_{Ni}$  can be estimated by Eq. (8).

130 
$$\mathcal{G}_{Ni} = \frac{1}{1.20} \sqrt{1 + \frac{8.33g_e(n-i+1)}{(\rho_i + \rho'_i)\sqrt{f_{2,i}}}}$$
(8)

According to the Elastic-Perfectly-Plastic (EPP) model established by Tomazevic (2007) and Magenes et al. (1997), the yield strength is 0.9 times of the average ultimate capacity, so the storey yield strength coefficient of the  $i^{\text{th}}$  storey can be estimated as

134  $\xi_i = \frac{0.9R_{ui}}{V_i} = \frac{0.9A_{w,i}f_{vE,mi}}{V_i}$ (9)

135 Substitute Eq.  $(2) \sim$  Eq. (5) and Eq. (8) into Eq. (9), and the storey yield strength 136 coefficient can be further obtained by

137 
$$\xi_{i} = \frac{0.11\rho_{i}}{\alpha g_{e}} \cdot \frac{n+1}{(n+i)(n-i+1)} \sqrt{f_{2,i} + \frac{8.33g_{e}(n-i+1)\sqrt{f_{2,i}}}{\rho_{i} + \rho_{i}'}}$$
(10)

139 
$$\xi_{1} = \frac{0.11\rho_{1}}{\alpha g_{e}n} \sqrt{f_{2,1} + \frac{8.33ng_{e}\sqrt{f_{2,1}}}{\rho_{1} + \rho_{1}'}}$$
(11)

For single-storey masonry structures, the coefficient of 0.11 in Eq. (10) and Eq. (11) should be changed to  $0.11 \times 0.85 = 0.094$ .

143 
$$R = 1/\xi_{i,\min}$$
 (12)

144 where  $\xi_{i,\min}$  is the minimum yield strength coefficient of each storey.

145 If the bottom storey is the soft-storey, the strength reduction factor can be estimated as

146 
$$R = \frac{1}{\xi_1} = \frac{\alpha n g_e}{0.11 \rho_1 \sqrt{f_2 + \frac{8.33 n g_e \sqrt{f_2}}{\rho_1 + \rho'_1}}}$$
(13)

147 where  $f_2$  is the compressive strength of mortar of the bottom storey.

### 148 2.2 The yield displacement demand of the equivalent single-degree-of-freedom

Based on the roof displacement method for calculating the natural period of the structure and the displacement calculation method for the multi-limb wall and wall frame, and considering the influence of the bending deformation, shear deformation and coupling of the wall limbs, Jiang et al. (2018) proposed the calculation formula for the natural period  $T_{0,e}$  of masonry structures, which can be calculated as

154 
$$T_{0,e} = \left(0.132 + 0.050 \frac{H}{B}\right) \sqrt{\frac{g_e}{f_m^{1.5} h\rho}} \cdot H$$
(14)

155 Where, *H* and *B* are the total height and width of masonry structure, respectively; *h* is the soft-156 storey height;  $f_m$  is the compressive strength of masonry.

157 The elastic spectral displacement  $S_{de}$  of the equivalent single-degree-of-freedom (SDOF) 158 system is computed as (Fajfar 1999).

159 
$$S_{de} = \frac{T_{0,e}^2}{4\pi^2} \cdot \alpha g \tag{15}$$

160 The yield spectral displacement  $S_{dy}$  of the SDOF system is computed as (Fajfar 1999).

161 
$$S_{dy} = \frac{S_{de}}{R} = \frac{T_{0,e}^2}{4\pi^2} \cdot \frac{\alpha g}{R}$$
(16)

162 Eq. (13) is substituted into Eq. (16) to obtain the yield spectral displacement demand  $S_{dy}$ 163 (Eq. (17)).

164 
$$S_{dy} = 0.027 \cdot T_{0,e}^2 \cdot \frac{\rho_1}{ng_e} \sqrt{f_2 + \frac{8.33ng_e\sqrt{f_2}}{\rho_1 + \rho_1'}}$$
(17)

#### 165 **2.3 The inelastic displacement demand of the equivalent SDOF**

166 The equivalent linearization method can be used to estimate the inelastic displacement 167 demand of existing structures subjected to earthquakes. It was adopted in the capacity spectrum 168 method of ATC-40. In this method, the displacement demand of a structure can be determined 169 by the displacement demand of an equivalent linear system with an equivalent period and 170 equivalent damping. The evaluation targets including equivalent period and equivalent damping are functions of the ductility coefficient, so an iterative process is employed to 171 172 determine the displacement demand of existing structures. Meanwhile, an underestimate of 173 displacement demand of existing structures may result from the equivalent damping which is 174 independent of the natural period in the capacity spectrum method of ATC-40. To solve the 175 above problems, an equivalent linear system based on the secant period was proposed by Lin 176 and Lin (2009). In this method, the equivalent period and equivalent damping of the equivalent 177 linear system are functions of the strength reduction factor. Since the strength reduction factor 178 is known, iteration in determining the response of structures can be avoided effectively.

When the secant stiffness of the maximum displacement point is taken as the equivalent stiffness of the elastic-plastic model, the equivalent elastic period  $T_{eq}$  is calculated as (Borzi et al. 2001)

182 
$$T_{eq} = T_{0,e} \sqrt{\frac{\mu}{1 + \alpha_s (\mu - 1)}}$$
(18)

183 where  $\alpha_s$  is the post-yield stiffness;  $\mu$  is the ductility factor which is the ratio of the maximum 184 displacement to the yield displacement.

Base on the *R*- $\mu$ -*T* relationship, the  $\mu$  in Eq. (18) can be replaced by the strength reduction factor *R*. According to the *R*- $\mu$ -*T* relationship proposed by Newmark and Hall (1973),  $\mu = (R^2+1)/2$  can be substituted into Eq. (18) in short period region, while  $\mu = R$  can be substituted into Eq. (18) in long period region. For M-A sequences, Zhai et al. (2015) and Zhang et al. (2017; 2020) express the ductility factor  $\mu$  with strength reduction factor *R* through the *R*- $\mu$ -*T* relationship, these expressions clearly indicate the influence of different aftershock intensity. In this manuscript, the strength reduction factor *R* is computed by Eq. (19).

192 
$$R = 1 + \frac{a_0 \left( a_1 T_{0,e} + T_{0,e}^2 \right) \left( a_4 + \mu \right)}{\left( 1 + a_2 T_{0,e} + a_3 T_{0,e}^2 \right) \left( 1 + a_5 \mu \right)} \frac{1}{0.87 + 0.08 e^{1.2\gamma}}$$
(19)

193 where  $\gamma$  is the relative intensity of aftershock defined as the ratio of the peak ground 194 acceleration of the aftershock (PGA<sub>as</sub>) to that of the mainshock (PGA<sub>ms</sub>);  $a_0$ ,  $a_1$ ,  $a_2$ ,  $a_3$ ,  $a_4$  and 195  $a_5$  are regression parameters depending on the site classes as listed in Table 1. The site classes 196 are classified according to  $V_{20}$  referring to *Code for seismic design of buildings* (GB50011–

197 2010), and the corresponding  $V_{30}$  ranges are also listed in Table 1.  $\mu$  can be calculated according 198 to the inverse function of Eq. (19).

	The value of $u_0$ $u_3$							
Parameter	$V_{20}$	$V_{30}$	$a_0$	$a_1$	$a_2$	$a_3$	$a_4$	<i>a</i> 5
Site class I	$V_{20} > 500 \text{m/s}$	$V_{30}$ > 596m/s	0.86	10.83	9.68	0.57	-0.79	0.02
Site class 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
Site class III	$150 \text{m/s} < V_{20} \le 250 \text{m/s}$	$158 \text{m/s} < V_{30} \leq 278 \text{m/s}$	1.03	10.93	11.49	0.77	-0.95	0.04
Site class IV	$V_{20} \leqslant 150 \mathrm{m/s}$	$V_{30} \leqslant 158$ m/s	0.66	13.25	9.95	0.55	-0.81	0.01
00								

199 **Table 1** The value of  $a_0 \sim a_5$ 

201 Considering the effect of M-A sequences on elastic spectra, the maximum seismic

202 influence coefficient  $\alpha_{max}$  of M-A sequences can be expressed as  $\alpha_{max}=2.25 \cdot PGA_{ms} \cdot (1+0.03 \cdot \gamma)$ 203

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

(20)

(Zhang 2020).

204 The equivalent damping  $\zeta_{eq}$  of the EPP model is calculated as (Lin and Lin 2009)

206 where  $\zeta_0$  is the inherent damping.

207 According to the General rule for performance-based seismic design of buildings (CECS 208 160: 2004) (2004), the damping reduction factor of the EPP model can be obtained by Eq. (21).

209
$$B = \begin{cases} 1 + \frac{0.05 - \zeta_{eq}}{0.06 + 1.4\zeta_{eq}} & (T_{eq} \le T_g) \\ \left(1 + \frac{0.05 - \zeta_{eq}}{0.06 + 1.4\zeta_{eq}}\right) \cdot \left(\frac{T_g}{T_{eq}}\right)^{0.9 + \frac{0.05 - \zeta_{eq}}{0.5 + 5\zeta_{eq}}} & (T_{eq} > T_g) \end{cases}$$
(21)

210 where  $T_g$  is the characteristic period of ground motion.

211 The inelastic spectral displacement  $S_{dp}$  of the SDOF system is calculated as (Fajfar 1999)

212 
$$S_{dp} = \frac{T_{0,e}^2}{4\pi^2} \cdot \alpha g \cdot C = \frac{T_{eq}^2}{4\pi^2} \cdot \alpha g \cdot B$$
(22)

213 where, C is the inelastic displacement amplification factor, which can be estimated as

214 
$$C = \left(\frac{T_{eq}}{T_{0,e}}\right)^2 \cdot B$$
(23)

The analysis shows that for the masonry structures built on site class III and site class IV, the condition of  $T_{eq} \le T_g$  is generally satisfied. Therefore, the inelastic displacement amplification factor *C* (Eq. (24)) is derived by substituting Eq. (18) and Eq. (21) into Eq. (23).

218 
$$C = \frac{R^2 + 1}{2} \cdot \left( 1 - \frac{0.079 T_{0,e}^{-0.252} \sqrt{R - 1}}{0.13 + 0.1106 T_{0,e}^{-0.252} \sqrt{R - 1}} \right)$$
(24)

#### 219 2.4 The storey drift demand of masonry structures

Both earthquake damage investigations and shaking table tests have shown that the most common failure mode of masonry structures subjected to earthquakes is soft-storey failure mechanism caused by the shear failure of the walls between windows. The deformation mainly concentrates on a critical storey, which is generally the bottom storey, when the storey stiffness is relatively uniform (Wang 2008; Tomaževič and Weiss 2010). Therefore, the storey shear model is adopted to determine the deformation of masonry structures, while the soft-storey yielding mechanism is applied (Borzi et al. 2008).

Assuming that the vibration mode of masonry structures remains linear (inverted triangle) before yielding (Borzi et al. 2008), the yield displacement demand  $\delta_y$  of the soft storey is calculated as

 $\delta_{y} = \frac{h}{\Gamma_{x} H} S_{dy}$ (25)

231 where  $\Gamma_h$  is the modal height coefficient.  $\Gamma_h$  for a regular distributed mass is approximately 232 0.67.

For a vertically irregular masonry structure, it is assumed that its inelastic displacement is entirely generated by the soft storey. Thus, the inelastic displacement demand  $\delta_p$  of the soft storey is calculated as (Priestley et al. 2007)

236 
$$\delta_p = \delta_y + (S_{dp} - S_{dy}) \tag{26}$$

For a vertically regular masonry structure, assuming that the inelastic displacement is mostly generated by the soft storey with a small part generated by the adjacent storeys, the inelastic displacement demand  $\delta_p$  of the soft storey is calculated as (Restrepo-Velez 2003)

240 
$$\delta_{p} = \delta_{y} + \frac{S_{dp} - S_{dy}}{0.8 + 0.1n}$$
(27)

According to Eq. (26) and Eq. (27), the maximum storey drift  $\theta_{max}$  of masonry structures can be obtained by Eq. (28).

$$\theta_{\max} = \delta_p / h \tag{28}$$

244 The calculation flowchart of the  $\theta_{max}$  for unreinforced masonry structures subjected to M-

A sequences is shown in Figure 1.





248

246

243

## 249 **3. Validation of finite element model**

The effectiveness of the method proposed in this manuscript is verified through a masonry residential building. The residential building model is a 5-storey masonry structure with a storey height of 3.0m, a width of 9.3m, and a length of 39.6m, as shown in Figure 2. The thicknesses of all masonry walls are 240mm, and the anti-seismic wall area ratio  $\rho'$  in the transverse direction and  $\rho$  in the longitudinal direction are 0.084 and 0.049 respectively. The combined gravity load per unit building area  $g_e$  is 1.0 Dead load + 0.5 Live Load = 11.0 kN/m<sup>2</sup>, in which the dead load is the sum of the gravity load of the floor (4.0 kN/m<sup>2</sup>) and the gravity load of the masonry walls (6.0 kN/m<sup>2</sup>), and the live load is 2.0 kN/m<sup>2</sup>. The compressive strength of brick clay and mixed mortar adopted in the current study are 10.0MPa and 2.0MPa, respectively. The compressive strength of masonry is 2.81MPa.

Initially, a part of the structure was modelled (the shaded part of the masonry structure shown in Figure 2). The correctness of the modeling method is verified by comparing the simulation results with the shaking table test results. Then, the whole structure is modelled (the whole masonry structure shown in Figure 2) to calculate the storey drift responses under M-A sequences. Finally, the results of the proposed method are compared with those obtained from the numerical simulations to verify the effectiveness of the proposed method.



267 **Figure 2.** Plane of the unreinforced masonry structure model (unit: mm).

268

266

## 269 **3.1 Validation of finite element model**

270 The macro-modeling method simplifies the masonry into a homogenous material, and the 271 mechanical properties of the homogenous material are determined by both the bricks and the 272 mortar. The macro-modeling method ignores the difference of mechanical properties between 273 bricks and mortar, as well as their interaction. Although the local behaviors of the masonry, 274 such as crack localization and joint opening, are difficult to reproduce, satisfactory results can 275 be obtained for the global responses and damage distribution with a low computational cost. 276 Figure 3(a) illustrates the finite element model as a part of the whole structure as discussed in 277 Figure 2. The model consists of masonry walls and reinforced concrete floor slabs. Multilayer 278 shell elements with reduced integration were applied to simulate the nonlinear behavior of the 279 masonry walls and the reinforced concrete floor. For the constitutive laws, the kinematic

- 280 hardening model of the steel and the plasticity model of masonry and concrete with damage
- 281 energy consumption were considered. The specific modeling method can be found elsewhere
- 282 (Zhang and Wang 2013).



**Table 2**. The comparison between numerical and experimental results.

Direction	Measured	Analytical	Numerical	Analytical	Numerical
	natural period/s	natural period/s	natural period /s	error/%	error/%
X-direction	0.397	0.395	0.394	-0.50	-0.76

297

298 El Centro wave and Taft wave were selected to study the displacement response of masonry 299 structure. The comparison of roof displacement between the numerical results and the 300 experimental results is shown in Figure 4. The roof displacement of the numerical results has 301 taken the similarity coefficient 4:1 into consideration. The results show that the numerical 302 curves are basically consistent with the experimental curves, the maximum top displacement 303 of the numerical simulation is close to the maximum displacement of the experimental 304 measurement, and the error is within 15%. Therefore, the numerical model can be used to study 305 the seismic response of unreinforced masonry structures subjected to M-A sequences.





314 **3.2** Comparison between finite element method and the proposed method



To accurately obtain the response of masonry structure subjected to M-A sequences, only real earthquake records are selected. Based on the selection principles proposed in previous research (Shen et al. 2019), 8 M-A sequence records for site class II are chosen from different earthquake events to consider earthquake uncertainty and listed in Table 3.

The magnitude of the mainshock in the actual M-A sequence is greater than that of the aftershock, so PGA<sub>ms</sub> is generally not less than PGA<sub>as</sub>. To study the impact of the relative intensity of aftershocks,  $\gamma$  is set to 0, 0.5, 0.8, and 1.0, respectively ( $\gamma$ =0 indicates mainshock only).

Seismograph station	M-A	M-A Time		$M_{\rm w}$
Managua ESSO	Mainshock	1972/12/23 06:29	0.372	6.2
Managaa 1990	Aftershock	1972/12/23 07:19	0.263	5.2
Long Valley Dam	Mainshock	1980/05/25 16:34	0.430	6.0
Long valley Dam	Aftershock	1980/05/25 16:49	0.482	5.7
Kalamata	Mainshock	1986/09/13 17:25	0.235	6.2
Txututtuuu	Aftershock	1986/09/15 11:41	0.241	5.4
I A - Obregon Park	Mainshock	1987/10/01 14:42	0.428	6.0
LA Oblegon Funk	Aftershock	1987/10/04 10:59	0.344	5.3
LA - Century City	Mainshock	1994/01/17 12:31	0.256	6.7
CC North	Aftershock	1994/01/17 12:32	0.162	6.1
CHV029	Mainshock	1999/09/20 17:47	0.277	7.6
011102)	Aftershock	1999/09/20 17:57	0.241	5.9
GRAN SASSO	Mainshock	2009/04/06 01:33	0.145	6.3
010111 011000	Aftershock	2009/04/07 17:47	0.252	5.6
CHB005	Mainshock	2011/03/11 13:46	0.180	9.0
CIID005	Aftershock	2011/03/11 15:15	0.175	7.7
	Seismograph station Managua ESSO Long Valley Dam Kalamata LA - Obregon Park LA - Century City CC North CCHY029 GRAN SASSO	Seismograph stationM-AMainshockAftershock <t< td=""><td>Seismograph stationM-ATimeManagua ESSOMainshock1972/12/23 06:29Aftershock1972/12/23 07:19Aftershock1972/12/23 07:19Hainshock1980/05/25 16:34Aftershock1980/05/25 16:49Aftershock1980/05/25 16:49Mainshock1980/05/25 16:49Aftershock1980/05/25 16:49Aftershock1980/05/25 16:49Aftershock1980/05/25 16:49Aftershock1986/09/13 17:25Aftershock1986/09/15 11:41Aftershock1987/10/01 14:42Aftershock1987/10/04 10:59IAA- Obregon ParkMainshockAftershock1994/01/17 12:31CC NorthMainshockAftershock1999/09/20 17:47Aftershock1999/09/20 17:57Aftershock1999/09/20 17:47Aftershock2009/04/06 01:33Aftershock2009/04/07 17:47Aftershock2009/04/07 17:47Aftershock2011/03/11 13:46Aftershock2011/03/11 15:15</td><td>Seismograph stationM-ATimePGAManagua ESSOMainshock1972/12/23 06:290.372Aftershock1972/12/23 07:190.263Long Valley DamMainshock1980/05/25 16:340.430Aftershock1980/05/25 16:490.482KalamataMainshock1986/09/13 17:250.235KalamataMainshock1986/09/15 11:410.241LA - Obregon ParkMainshock1987/10/01 14:420.428LA - Century CityMainshock1987/10/04 10:590.344CC NorthAftershock1994/01/17 12:310.256CC NorthAftershock1999/09/20 17:470.277CHY029Mainshock1999/09/20 17:570.241GRAN SASSOAftershock2009/04/06 01:330.145CHB005Mainshock2011/03/11 13:460.180CHB005Mainshock2011/03/11 13:460.180</td></t<>	Seismograph stationM-ATimeManagua ESSOMainshock1972/12/23 06:29Aftershock1972/12/23 07:19Aftershock1972/12/23 07:19Hainshock1980/05/25 16:34Aftershock1980/05/25 16:49Aftershock1980/05/25 16:49Mainshock1980/05/25 16:49Aftershock1980/05/25 16:49Aftershock1980/05/25 16:49Aftershock1980/05/25 16:49Aftershock1986/09/13 17:25Aftershock1986/09/15 11:41Aftershock1987/10/01 14:42Aftershock1987/10/04 10:59IAA- Obregon ParkMainshockAftershock1994/01/17 12:31CC NorthMainshockAftershock1999/09/20 17:47Aftershock1999/09/20 17:57Aftershock1999/09/20 17:47Aftershock2009/04/06 01:33Aftershock2009/04/07 17:47Aftershock2009/04/07 17:47Aftershock2011/03/11 13:46Aftershock2011/03/11 15:15	Seismograph stationM-ATimePGAManagua ESSOMainshock1972/12/23 06:290.372Aftershock1972/12/23 07:190.263Long Valley DamMainshock1980/05/25 16:340.430Aftershock1980/05/25 16:490.482KalamataMainshock1986/09/13 17:250.235KalamataMainshock1986/09/15 11:410.241LA - Obregon ParkMainshock1987/10/01 14:420.428LA - Century CityMainshock1987/10/04 10:590.344CC NorthAftershock1994/01/17 12:310.256CC NorthAftershock1999/09/20 17:470.277CHY029Mainshock1999/09/20 17:570.241GRAN SASSOAftershock2009/04/06 01:330.145CHB005Mainshock2011/03/11 13:460.180CHB005Mainshock2011/03/11 13:460.180

324 **Table 3**. Selected ground motion record of M-A sequence

325

#### 326 3.2.2 The effectiveness of the proposed method

Using the validated modeling method introduced in Section 3.1, a finite element model of the whole structure is established based on the masonry structure shown in Figure 2 to verify the effectiveness of the proposed method. The comparison between analytical and experimental results is shown in Table 2. The error between the analytical and experimental results is 0.50%. The average storey drift of masonry structures under 8 M-A sequences was

- analyzed by the finite element method, and the storey drift of the masonry structure for site
- 333 class II is also calculated by the proposed method. The comparison of the results is shown in
- Figure 5. For  $PGA_{ms} = 0.1g$ , the errors between the numerical results and the analytical results
- are within 11.0%. For  $PGA_{ms} = 0.2g$ , the errors between the numerical results and the analytical
- results are within 8.0%. It appears that the analytical results are in a good agreement with the
- 337 numerical results.



338 339 **Figure 5.** Comparison of analytical and numerical results.

#### 341 4. Seismic analysis of unreinforced masonry structures subjected to M-A sequences

The 5-storey masonry structure shown in Figure 2 was used as a basic structure model to study the effect of M-A sequences on the seismic response of unreinforced masonry structures. The  $\theta_{max}$  of the masonry structure for four site classes subjected to M-A sequences with  $\gamma = 0$ , 0.5, 0.8 and 1.0 are shown in Figure 6. As shown in Figure 6, the  $\theta_{max}$  of the masonry structure shows the same trend of variation with increasing mainshock intensity regardless of the site class and the aftershock intensity.

For a given site class and PGA<sub>ms</sub>, the  $\theta_{max}$  of the masonry structure increases with increasing  $\gamma$ . For site class II and PGA<sub>ms</sub>=0.2g, the  $\theta_{max}$  of the masonry structure subjected M-A sequences with  $\gamma = 0, 0.5, 0.8$  and 1.0 is 0.417%, 0.448%, 0.472%, and 0.491%, respectively, indicating that the aftershock can lead to a larger storey drift of masonry structures.

The performance level of a generic masonry structure is usually defined by roof displacement or storey drift. According to the research on the relationship between the performance level and the  $\theta_{max}$  of masonry structures, three performance levels, namely *Light damage limit state* (LS1), *Significant damage limit state* (LS2), and *Collapse limit state* (LS3), are employed to describe the structural damage states. An average drift of 0.130%, 0.340%, and 0.720% can be used to identify the LS1, LS2, and LS3 limit conditions of unreinforced masonry structures (Borzi et al. 2008). For site class II, when the  $\theta_{max}$  of the masonry structure reaches 0.720%, the PGA<sub>ms</sub> of the M-A sequence with  $\gamma = 0$ , 0.5, 0.8, and 1.0 is 0.25g, 0.24g, 0.23g, and 0.23g, respectively, indicating that the larger the aftershock intensity is, the earlier

361 the masonry structure reaches the limit state.

362



Figure 6. The  $\theta_{max}$  of the 5-storey masonry structure for different site classes and M-A sequences with different  $\gamma$ : (a) site class I, (b) site class II, (c) site class III, (d) site class IV.

369

#### **4.1 Effect of site class**

To assess the effect of site classes on the seismic response of masonry structures, the  $\theta_{\text{max}}$ of the reference structure model on different site classes is normalized by the mean  $\theta_{\text{max}}$  of all 373 site classes, respectively. In this way, the error of the  $\theta_{max}$  without considering site conditions 374 can be studied quantitatively. The normalized  $\theta_{max}$  of the reference masonry structure model 375 for the different site classes and M-A sequences ( $\gamma = 0.5, 0.8, \text{ and } 1.0$ ) is shown in Figure 7. Structures founded on site class I exhibit a lower  $\theta_{max}$  value. This indicates that  $\theta_{max}$  can be 376 377 overestimated up to 19.2% for site class I if site class effect is ignored. Structures founded on 378 site class II and site class III exhibit a higher  $\theta_{max}$  value, indicating that site class effect can 379 lead to underestimation of  $\theta_{\text{max}}$  on site class II and site class III up to 4.8% and 17.6%, 380 respectively.





388 **4.2 Effect of the number of storeys** 

In order to investigate the effect of the number of storeys, n, on the seismic response of masonry structures, the n of the reference structure model is set to 1, 2, 3, 4, and 5 respectively,







Figure 8 shows that the  $\theta_{max}$  of masonry structures with different *n* increase evidently with the increase of *n*. For PGA<sub>ms</sub>=0.2g,  $\gamma$ =0, and site class II, the  $\theta_{max}$  of masonry structures with *n*=1, 2, 3, 4, and 5 are 0.009%, 0.027%, 0.072%, 0.197%, and 0.417%, respectively. For PGA<sub>ms</sub>=0.2g,  $\gamma$ =1.0 and site class II, the  $\theta_{max}$  of masonry structures with *n*=1, 2, 3, 4, and 5 are

(b) site class II, (c) site class III, (d) site class IV.

410 0.010%, 0.028%, 0.084%, 0.233%, and 0.491%, respectively. The results indicate that the n 411 has a significant effect on the  $\theta_{max}$  of masonry structures. The smaller the *n*, the greater the 412 PGA<sub>ms</sub> required for the masonry structure to enter the inelastic phase. The reason is that with 413 the decrease of n, the anti-overturning requirements of the structure decrease, and the base 414 shear force of the structure also decreases, that is, the plane layout and material strength of the 415 1-storey masonry structure and the 1<sup>st</sup> storey of the 5-storey masonry structure are completely 416 consistent, both have the same seismic capacity, but the seismic shear load of the former is 417 significantly less than that of the latter, resulting in the high-rise masonry structure entering the 418 plastic phase with a smaller PGA<sub>ms</sub>. It should be pointed out that the plane layout and material 419 strength of the masonry structures with different n in this manuscript are consistent, so as to 420 directly compare the effects of *n*. However, the anti-seismic wall area ratio  $\rho$  and the material 421 strength of the actual low-rise masonry structure is generally smaller than that of the low-rise 422 masonry structure in this manuscript, resulting in the seismic capacity of the former being 423 smaller than that of the latter, that is, the low-rise masonry structure may damage in smaller 424 PGA<sub>ms</sub> in practice.

Earthquake damage investigations have showed that the damage degree of masonry structures is directly proportional to the n of masonry structures in the same intensity zone (Zhou 2011). The results of this manuscript are consistent with the earthquake damage investigation. Therefore, for high rise masonry structures, seismic strengthening (such as RC tie columns, ring beams etc.) and materials with higher strength must be adopted to meet the seismic requirements (Zhang et al. 2021).

431 To compare the effects of M-A sequences with different  $\gamma$ , the  $\theta_{max}$  of masonry structures with  $\gamma = 0.5$ , 0.8 and 1.0 is normalized by the  $\theta_{max}$  of the reference structure with  $\gamma = 0$ , as 432 433 shown in Figure 9. Figure 9 indicates that the  $\theta_{max}$  for M-A sequences was quite close to the 434  $\theta_{\text{max}}$  for mainshock for a range of PGA<sub>ms</sub> less than 0.31g, 0.20g, 0.16g, 0.13g, and 0.11g when 435 n=1, 2, 3, 4, and 5, respectively. The reason is that the masonry structures behave elastically in 436 the  $PGA_{ms}$  range, and the structural response mainly depends on the elastic spectra, but the 437 difference of elastic spectra with different  $\gamma$  is small. Therefore, the effect of aftershocks can 438 be ignored in elastic phase.



443 **Figure 9.** The  $\theta_{\max,\gamma}/\theta_{\max,\gamma=0}$  of masonry structures for different  $\gamma$  and different *n*, site class II: 444 (a)  $\gamma = 0.5$ , (b)  $\gamma = 0.8$ , (c)  $\gamma = 1.0$ .

445 The  $\theta_{max}$  for M-A sequences is significantly greater than that for only mainshock when 446 PGA<sub>ms</sub> is larger than 0.31g, 0.20g, 0.16g, 0.13g, and 0.11g for n=1, 2, 3, 4, and 5, respectively. 447 The reason is that the masonry structures enter inelastic phase under the strong mainshock, and 448 the damage degree of the structure is further aggravated due to aftershock energy. For  $\gamma=0.5$ , 449 the  $\theta_{\text{max}}$  of masonry structure subjected to M-A sequences increases by 7.3%, indicating that the effect of aftershock with  $\gamma$  less than 0.5 can be negligible. For  $\gamma=0.8$  and 1.0, the  $\theta_{max}$  of 450 451 masonry structure subjected to M-A sequences increases by 13.1% and 19.0%, respectively. 452 The result shows that the effect of aftershocks with  $\gamma$  more than 0.8 is significant and cannot 453 be negligible.

454 The plane layout of masonry structures can be reflected by the anti-seismic wall area ratio 455  $\rho$ , which indicates the ratio of the total anti-seismic wall area at the 1/2-storey height to the 456 storey area of the structure. To study the effect of  $\rho$  on the structural response, the  $\rho$  of the 457 reference structure model is set to 0.021, 0.035, and 0.049, respectively. The  $\theta_{\text{max}}$  of the 458 masonry structures subjected to M-A sequence are calculated, and the results are shown in 459 Figure 10.

460



463 **Figure 10.** The  $\theta_{\text{max}}$  of masonry structures for different  $\rho$  and different *n*, site class II: (a)  $\gamma =$ 464 0, (b)  $\gamma = 1.0$ .

465

461 462

466 For PGA<sub>ms</sub>=0.2g and  $\gamma = 0$ , the  $\theta_{max}$  of the 5-storey masonry structures with  $\rho = 0.021$ , 467 0.035, and 0.049 are 0.971%, 0.536% and 0.417%, respectively. At the same seismic intensity, 468 the  $\theta_{\text{max}}$  of the 2-storey masonry structures with  $\rho = 0.021, 0.035, \text{ and } 0.049$  are 0.358%, 469 0.100% and 0.027%, respectively. For PGA<sub>ms</sub>=0.2g and  $\gamma = 1.0$ , the  $\theta_{max}$  of the 5-storey 470 masonry structures with  $\rho = 0.021, 0.035, \text{ and } 0.049$  are 1.142%, 0.631% and 0.491%, 471 respectively. At the same seismic intensity, the  $\theta_{max}$  of the 2-storey masonry structures with  $\rho$ = 0.021, 0.035, and 0.049 are 0.419%, 0.115% and 0.028%, respectively. By decreasing the 472 473 anti-seismic wall ratio from 0.049 to 0.035, the  $\theta_{max}$  increases to 2.60 times on average, which 474 shows that the  $\theta_{\text{max}}$  of the masonry structures significantly increases with the decrease of  $\rho$ . 475 The reason is that the seismic load is borne by the masonry walls along the earthquake load, 476 and larger area of seismic wall will lead to greater shear capacity of the structure and smaller 477 structural response. For 2-storey masonry structures in rural areas in China, the value of  $\rho$  is 478 generally closer to 0.021. It can be seen from Figure 10 that the 2-storey masonry structure 479 with  $\rho = 0.021$  is seriously damaged when PGA<sub>ms</sub> = 0.2g, which is consistent with the earthquake damage investigation. 480

#### 482 **4.3 Effect of mortar strength**

483 Mortar strength  $f_2$  is an important factor affecting the shear capacity of masonry structures. 484 To study the effect of  $f_2$  on the  $\theta_{max}$  of masonry structures subjected to M-A sequences, the  $f_2$ 485 of the reference structure model is set to 1.0MPa, 2.5 MPa, 5.0 MPa, 7.5 MPa, and 10.0 MPa, 486 respectively, while other parameters of the reference structure model remain unchanged. The 487  $\theta_{max}$  of the masonry structures with different mortar strengths for the four site classes subjected 488 to M-A sequences with different  $\gamma$  are shown in Figure 11.

489 As shown in Figure 11, the  $\theta_{max}$  of masonry structures decreases as the mortar strength 490 increases. For PGA<sub>ms</sub> = 0.2g,  $\gamma$  = 0, and site class II, the  $\theta_{max}$  of 5-storey masonry structures 491 with  $f_2$  = 1.0MPa, 2.5 MPa, 5.0 MPa, 7.5 MPa, and 10.0 MPa are 0.648%, 0.348%, 0.160%, 492 0.083%, and 0.044%, respectively. For PGA<sub>ms</sub> = 0.2g,  $\gamma$  = 1.0, and site class II, the  $\theta_{max}$  of 5-493 storey masonry structures with  $f_2$ = 1.0MPa, 2.5 MPa, 5.0 MPa, 7.5 MPa, and 10.0 MPa are 494 0.764%, 0.411%, 0.191%, 0.092%, and 0.046%, respectively. Overall, the  $\theta_{max}$  of the masonry 495 structure with f<sub>2</sub>=2.5 MPa, 5.0 MPa, 7.5 MPa, and 10.0 MPa are 0.55, 0.25, 0.11, and 0.05 496 times of those with  $f_2=1.0$  MPa, respectively, indicating that the mortar strength has a great 497 influence on the structural response of masonry structures. The reason is that with the increase 498 of mortar strength, the shear capacity of masonry increases, resulting in less structural damage. 499 Therefore, the higher the mortar strength, the smaller the  $\theta_{max}$ , and the better the seismic 500 performance of the structures.





505 Figure 11. The  $\theta_{max}$  of masonry structures for different mortar strengths, PGA<sub>ms</sub>=0.2g: (a) site class I, 506 (b) site class II, (c) site class III, (d) site class IV.

503 504

508 For PGA<sub>ms</sub> = 0.2g, the  $\theta_{max,\gamma=1.0}/\theta_{max,\gamma=0}$  of the masonry structure with  $f_2$  = 1.0MPa, 2.5 MPa, 509 5.0 MPa, 7.5 MPa, and 10.0 MPa are 1.18, 1.18, 1.19, 1.11, and 1.04, respectively. For PGA<sub>ms</sub> 510 = 0.1g, the  $\theta_{\max,\gamma=1.0}/\theta_{\max,\gamma=0}$  of the masonry structure with  $f_2$  = 1.0MPa, 2.5 MPa, 5.0 MPa, 7.5 511 MPa, and 10.0 MPa are 1.12, 1.06, 1.05, 1.04, and 1.04, respectively. The results show that the 512  $\theta_{\text{max}}$  of masonry structures increase by 4.0% to 19.0%, which is consistent with the results of 513 Section 4.2. The masonry structure with  $f_2 = 1.0$  MPa has entered the inelastic phase when 514  $PGA_{ms} = 0.1g$ , and the masonry structure with  $f_2 = 10.0$  MPa behaves elastically when  $PGA_{ms} =$ 515 0.2g, indicating that the masonry structure tends to remain elastic for higher seismic loads with 516 the increase of mortar strength.

517

#### 518 **5. Summary and conclusions**

The seismic response of masonry structures subjected to M-A sequences was investigated involving various parameters such as the aftershock intensity, the anti-seismic wall area ratio, the site classes, the number of storeys, and the mortar strength by using a simplified method newly proposed. The main conclusions are summarized as follows:

523 (1) On the basis of the non-iterative equivalent linearization method and the soft-storey 524 failure mechanism of multi-storey masonry structures, an analytical method for the maximum 525 storey drift ( $\theta_{max}$ ) of masonry structures subjected to M-A sequences was proposed. There was 526 excellent agreement between analytical and numerical results for the  $\theta_{max}$  of masonry structures subjected to M-A sequences. The method proposed in this manuscript avoids iterativecalculation and as a result, has a small workload and is easy to implement.

529 (2) The  $\theta_{\text{max}}$  of masonry structures increases with the increase of aftershock intensity. The 530 effect of aftershocks on masonry structures in plastic phase is more distinct than that in elastic 531 phase. Furthermore, the effect of aftershock on the  $\theta_{\text{max}}$  of masonry structures can be ignored 532 when the relative intensity of the aftershock is less than 0.5, and the  $\theta_{\text{max}}$  of masonry structures 533 can increase by approximately 19.0% when the relative intensity of the aftershock equals 1.0.

534 (3) There is a significant variance for the  $\theta_{\text{max}}$  of masonry structures subjected to M-A 535 sequences on different site classes. The regardless of site class will lead to overestimation on 536 the  $\theta_{\text{max}}$  for site class I by 19.2% and underestimation on the  $\theta_{\text{max}}$  for site class III by 17.6%.

537 (4) With the increase of anti-seismic wall area ratio (indicating the ratio of the total anti-538 seismic wall area at the 1/2-storey height to the storey area of the structure), the  $\theta_{\text{max}}$  of masonry 539 structures subjected to M-A sequences decreases drastically. By decreasing the anti-seismic 540 wall ratio from 0.049 to 0.035, the  $\theta_{\text{max}}$  increases to 2.60 times on average.

541 (5) With increasing number of storeys, the  $\theta_{max}$  of masonry structures subjected to M-A 542 sequences increases drastically. As the number of storeys decreases, the anti-overturning 543 requirements and the base shear force of the masonry structures decrease, resulting in smaller 544  $\theta_{max}$  and less damage.

545 (6) The effect of mortar strength on the  $\theta_{max}$  of masonry structures subjected to M-A 546 sequences is significant. Overall, the  $\theta_{max}$  of the masonry structures with mortar strength equal 547 to 2.5 MPa, 5.0 MPa, 7.5 MPa and 10.0 MPa is 0.55, 0.25, 0.11, and 0.05 times of that with 548 mortar strength equal to 1.0 MPa, respectively.

549

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