Development of a Practical Approach for Modeling of Unreinforced Masonry Buildings

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ABSTRACT

At different parts of the world, there are many unreinforced masonry buildings have been constructed for different uses. Studies have shown the insufficiency in the design and performance of these buildings. Proper a modeling approach for nonlinear analyses of masonry buildings and seismic assessment of these buildings is necessary. This study presents a practical approach to model unreinforced masonry building. This approach considers the effects of including the lateral resistance of masonry walls in resisting seismic loads. The main aim of this article is showing the utility and efficiency of a simple method in the explanation of the behaviour of masonry buildings subjected to seismic loads. The simple models are essential because they provide easy and accessible understanding of the structural response. This approach was used to assess a common type of masonry building as a representative building of Iranian masonry schools. The results of static and dynamic nonlinear analyses show this approach offers a good balance between simplicity and accuracy and can be used for nonlinear analyses and seismic assessment of these buildings.

1. Introduction

The observed damage and collapse of confined masonry buildings during past earthquakes are partially due to lack of methodologies for estimating capacity of these buildings to withstand seismic loads [1]. For seismic assessment and retrofitting of these buildings it is necessary to present a modeling technique for nonlinear static and dynamic analyses. A number of studies were carried out on modeling and retrofitting of masonry building such as Taeko Tokoro et al. [2] who investigated the seismic performance of three reinforced masonry buildings with flexible diaphragms and evaluated the code criteria for determining the flexibility of the diaphragms. Bouchard [3] studied the performance-based approach to retrofit unreinforced masonry structures and compared FEMA 356 [4] approach results to a modified analysis. Eshghi and Pourazin [1] investigated in-plane behaviour of confined masonry walls-with and without- Opening and developed 2D finite element models of the walls and a pushover analysis carried out. Ahmad et al. [5] assessed the performance of low-rise confined masonry structures for earthquake ground motions using a probabilistic-based approach. In this approach, the masonry walls were idealized as one dimensional beam-column elements (wide-column

analogy) with bending and shear deformation. Marques and Lourenço [6] developed a model for pushover analysis of confined masonry structures. They presented a simplified numerical approach based on a wide-column approach to apply performance-based design procedures. This study presents a practical approach for modeling unreinforced masonry buildings for 3D nonlinear static and dynamic analyses. This approach, in the simplicity, applies all of FEMA 356 [4] criteria for modeling of unreinforced masonry building. As a case study, this paper is applied proposed approach to seismic assessment an unreinforced masonry building as a representative building derived based on a large survey of Iranian masonry schools in order to understand their behaviour during strong ground motions.

2. Modeling Process

The existing horizontal and vertical tie beams in masonry building were modeled such as beams and columns by using of Fiber section model. This study uses the OpenSees software [7] for modeling masonry building. Displacement-Based Beam-Column Element (fiber section model), developed by Taucer et al. [8] is one of the well-known tools for static and dynamic analysis of frames and reinforcement concrete shear walls. The masonry walls were included in the model by compression diagonal members (Figure 1-a). For this purpose, truss element in Opensees software [7] was used and behaviour curve was defined for each masonry walls. This paper applies FEMA 356 [4] generalized force-deformation relation for unreinforced masonry walls as shown in Figure (1-b). For each masonry walls an axial force-deformation relationship was defined.

Each masonry wall was considered as a spring and for each spring a behaviour curve was defined. The springs were modelled by truss element in Opensees software [7]. The material behaviour was defined such as generalized force-deformation relation as shown in Figure (1-b) based on FEMA 356 [4] and an axial force-deformation relationship was defined for the truss section. To define behaviour curve in Opensees software [7], Hysteretic material (Figure 2) was used, that defines several points for behaviour curve. As shown in Figure (1-b), points A,

B, C, D, E should be defined for each wall. In order to model piers between opening, behavior curve was defined for each pier, then incorporate them with Parallel spring command in Opensees [7] (Figure 3).
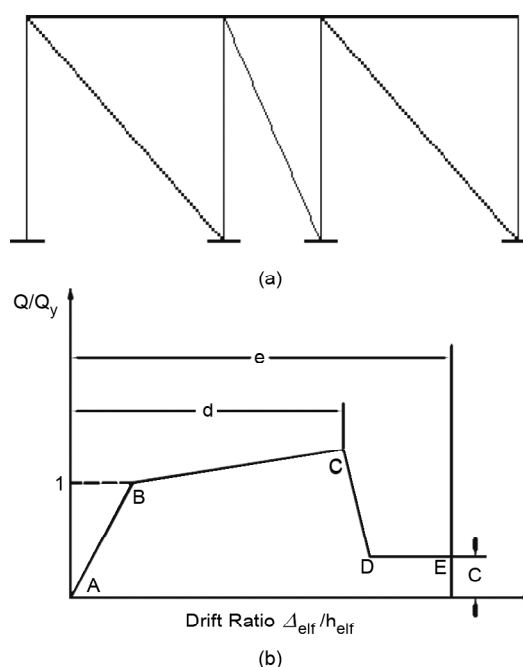

Figure 1. a) Modeling of horizontal and vertical tie beams and masonry wall as compression diagonal member, b) Generalized force-deformation relation [4].
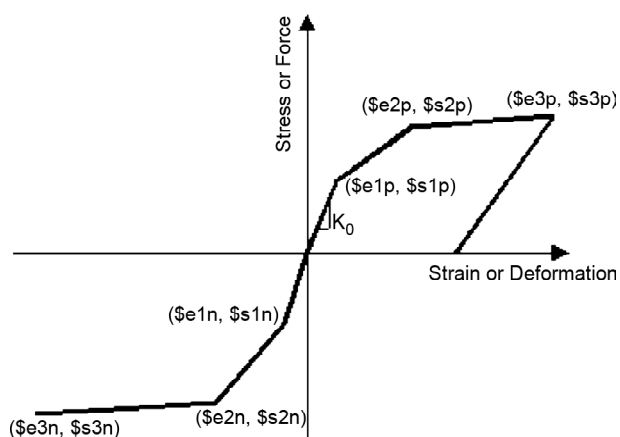

Figure 2. Hysteretic material in Opensees software [7].
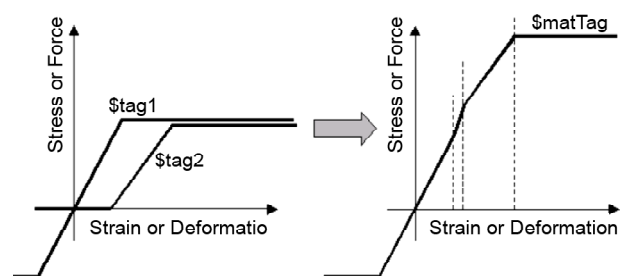

Figure 3. Incorporating Parallel springs in Opensees software [7].

The lateral stiffness of masonry walls determine based on FEMA 356 [4]. FEMA 356 [4] recommends two equations for lateral in-plan stiffness of masonry walls by considering flexural and shear deformations. This study used these equations to calculated stiffness of masonry walls. To determine in-plane strength of masonry walls, all the failure modes of masonry walls were considered. Failure modes for in-plane behaviour of unreinforced masonry based on FEMA 356 [4] are classified in four groups: (1) Rocking; (2) Bed-joint sliding; (3) Diagonal tension; and (4) Toe crushing. This guideline recommends equations for calculate theses failure modes, this study used these equations to calculated failure modes of masonry walls. Table 7-4 in FEMA 356 [4] gives values for deformation-controlled components that nonlinear force-deflection relations for deformation-controlled wall and pier components shall be established based on this information. This study applies all of FEMA 356 [4] requirements for modeling of unreinforced masonry building.

In this study, material model "concrete01" for concrete and "steel02" for steel were used. "Concrete01" has compression strength branch based on Modified Kent and Park model and no tensile strength [7]."Steel02" is used to construct the well-known nonlinear hysteric model of Giuffre-Menegotto-Pinto steel material [9] that includes isotropic strain hardening [7].

3. The Earthquake Ground Motion Records

The accelerograms that are used in the nonlinear dynamic analyses are selected based on a study [10] to reduce the scattering of dynamic analysis responses (Table 1). These ground motion records

were filtered using a Butterworth filter of order 8.

4. Representative Model Building

A typical low-rise confined masonry school (Figure 4) with horizontal and vertical ties and masonry walls was modeled into Opensees software [7] and studied by static and dynamic nonlinear analyses using the proposed modeling technique. The thickness of masonry walls in both east-west and north-south directions are 30 cm. Expected material properties were assumed as shown in Table (2).
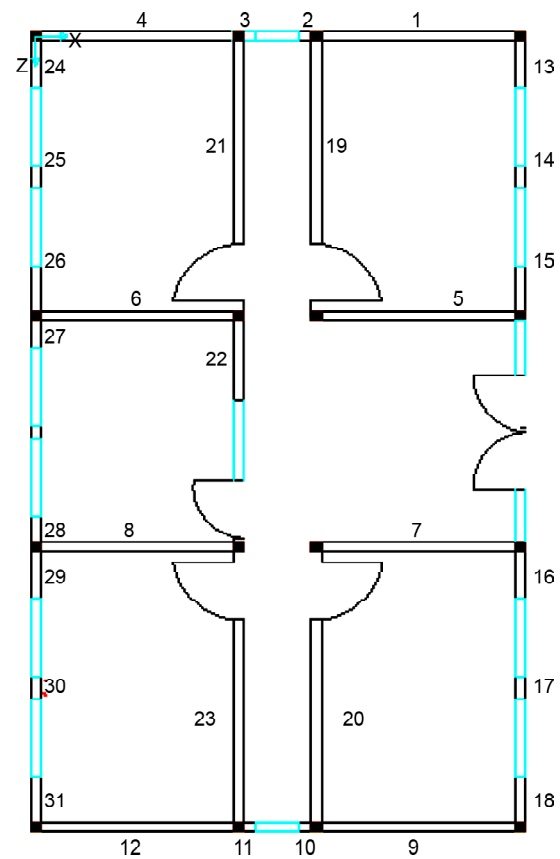

Figure 4. Building plan and its masonry walls.

Table 1. Property of accelerograms [10].

Event Number	Event	Label	Year	Station	M	PGA Processed (g)	Distance (Km)
Event 1	Bandar-e-Abas3	Bandar-e-Abas3*-C3	1975	Bandar-e-Abas3*	6.1	0.13	36
Event 2	Golbaft	Qazvin*-C3	1981	Qazvin*	7.4	0.27	94
Event 3	Avaj	Razan-C3	2002	Razan	6.5	0.20	35
Event 4	BINGOL	Np20030501002708_1201N-S	2003	BayindirlikveSkanMudurlugu Beverly Hills	6.3	0.50	12
Event 5	Northridge	Canyon Country-WLost cany-N90W	1994	Canyon Country-WLostcany	6.7	0.48	27
Event 6	Friuli	Friuli,Italy-Tolmezzo-NSX	1976	Tolmezzo	6.5	0.34	20
Event 7	Chi-Chi	Chi-Chi-CHY101-E	1999	Chi-Chi-CHY101	7.6	0.40	32
Event 8	Chi-Chi	Chi-Chi-TCU045-E	1999	Chi-Chi-TCU045	7.6	0.47	76

Table 2. Expected material properties.

Expected Shear Strength (kg/cm^2)	Expected Compressive Strength (kg/cm^2)	Elastic Modulus (kg/cm^2)	Shear Modulus (kg/cm^2)
1.5	32	17600	7040

Table 3. Properties of masonry wall 19.

Number of Similar Walls	Wall Height (m)	Wall Length (m)	I (cm^4)	A (cm^2)	K (ton/m)
4	2.8	6.55	6.5569	1.834	42971
Load Area (m^2)	Bed-Joint Sliding (ton)	Rocking (ton)	Diagonal Tension (ton)	Toe Compressive (ton)	Governing Mode
30.1	17.84	45.31	61.98	29.03	Bed-Joint Sliding

4.1. Model Properties

The engineering properties of masonry walls were calculated based on FEMA 356 [4] equations. For example, the stiffness and strength of masonry wall 19 were shown in Table (3). The Behavior curve of this masonry wall was defined, such as generalized force-deformation relation as shown in Figure (1-b) and is shown in Figure (5). As shown in Table (3) the governing mode of wall 19 is Bed-joint sliding.

$$\Delta_y = \frac{V_{BJS}}{K} = \frac{17.84}{42971} = 0.000415 \ m = 0.415 \ mm$$

Position of Point B: $\dfrac{\Delta_y}{h_{eff}} = \dfrac{0.000415}{2.8} = 0.000148$

Position of Point C: $\dfrac{\Delta_y}{h_{eff}} = 0.4\% = 0.004$

Position of Point E: $\dfrac{\Delta_y}{h_{eff}} = 0.8\% = 0.008$

To define the behaviour curve of other walls, the same approach as for wall 19 were followed.
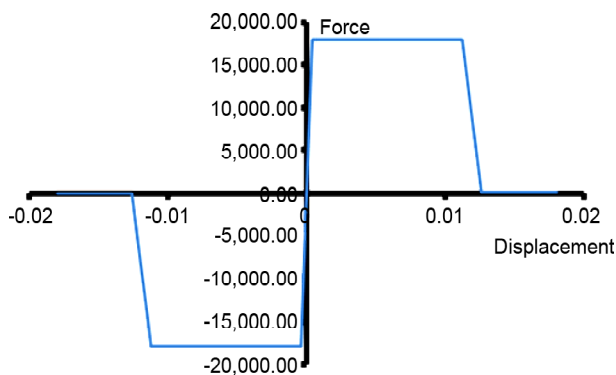
4.2. Nonlinear Static Analysis

For nonlinear static analysis, nonlinear force-deformation relationships should be generated for each of the wall and pier elements. These relationships are then used to develop a global force-deformation for the building. Figure (6) shows the response of masonry wall 19. This figure shows response of this wall was passed from Collapse Prevention and performance limit constitute a Collapse Prevention hazard.

Figure (7) shows the global force-deformation curve of masonry building in two directions, east-west and north-south. As shown in Figure (7), a sudden reduction occurred in the first part of these curves. These reductions demonstrate the point that lateral strength of existing masonry walls in the building are lost.

4.3. Nonlinear Dynamic Analysis

For nonlinear dynamic analysis, building model was subjected to eight accelerograms of Table (1). Figure (8) show time history drift response of masonry wall 19. Table (4) shows maximum response
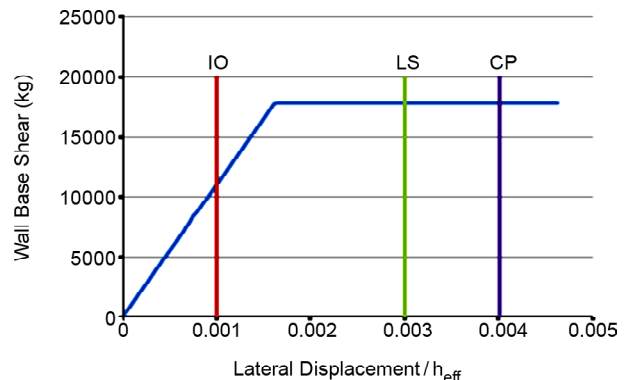

Figure 5. Behaviour curve of wall 19.

Figure 6. Base Shear-Lateral Displacement of masonry wall 19.

and mean of maximum response of wall 19 during eight earthquake ground motions. This table shows response of this wall was passed from Collapse Prevention and performance limit constitute a Collapse Prevention hazard.

Figure (9) shows the time history drift response of the building for north-south direction. Table (5) shows maximum drift of building and mean of

maximum response during eight earthquake ground motions for north-south direction.

Figure (10) show the time history drift response of building for east-west direction. Table (5) shows maximum drift of building and mean of maximum response during eight earthquake accelerograms for east-west direction. Figure (10-b) shows plastic deformation was occurred in this building. In some
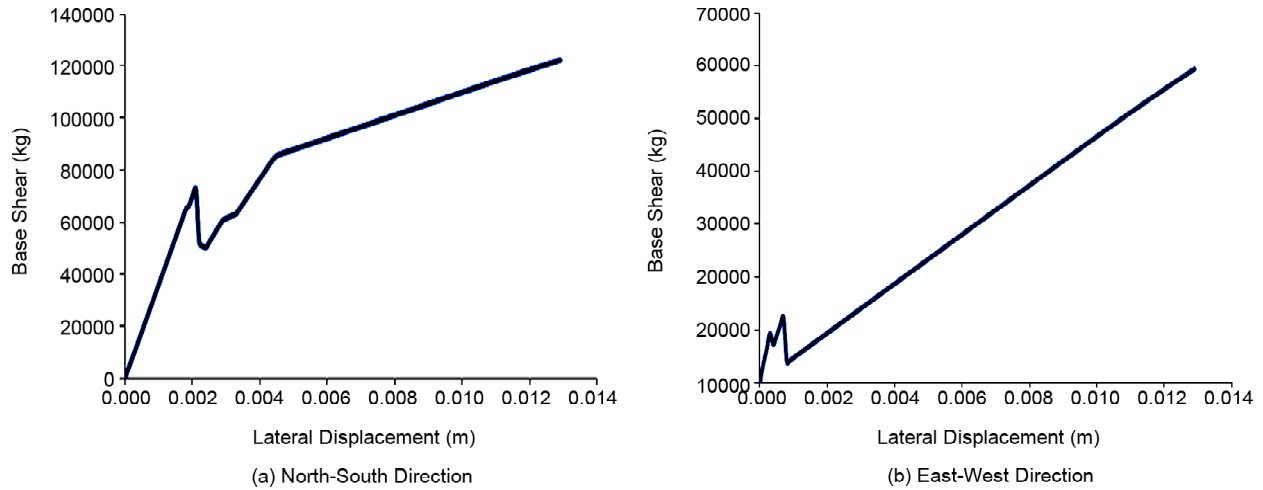

(a) North-South Direction

(b) East-West Direction

Figure 7. Global force-deformation curve from nonlinear static analysis.

(a) During Bandar-e-Abas3*-C3 Earthquake
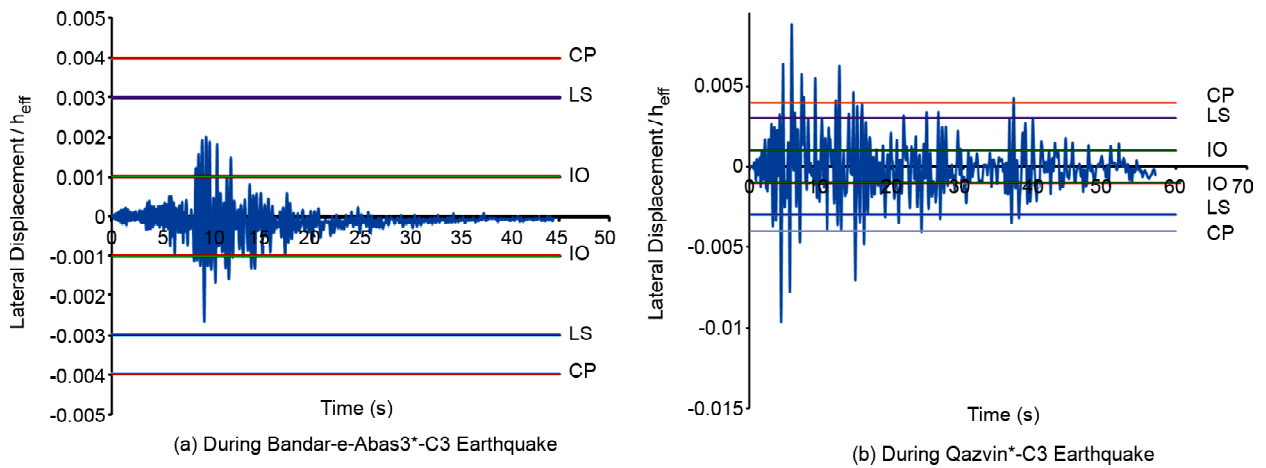

(b) During Qazvin*-C3 Earthquake

Figure 8. Time history drift response of masonry wall 19.

Table 4. Maximum response and mean of maximum response of wall19 during 8 earthquake accelerograms.

Mean of Max (Lateral Displacement/h_{eff})	Max (Lateral Displacement/h_{eff})								
	Event 8	Event 7	Event 6	Event 5	Event 4	Event 3	Event 2	Event 1	
0.0094	0.0166	0.006	0.0063	0.0174	0.0124	0.0042	0.0097	0.0026	Wall 19

Table 5. Maximum response and mean of maximum drift of masonry building during 8 earthquake accelerograms.

Mean of max (Lateral Displacement/h_{eff})	Max(Lateral Displacement/h_{eff})								
	Event 8	Event 7	Event 6	Event 5	Event 4	Event 3	Event 2	Event 1	
0.0083	0.0145	0.0053	0.0055	0.0153	0.0109	0.0037	0.0085	0.0023	North-South Dir.
0.013	0.0156	0.0162	0.0087	0.0233	0.0198	0.0063	0.0112	0.0031	East-West Dir.

(a) During Bandar-e-Abas3*-C3 Earthquake

(b) During Qazvin*-C3 Earthquake
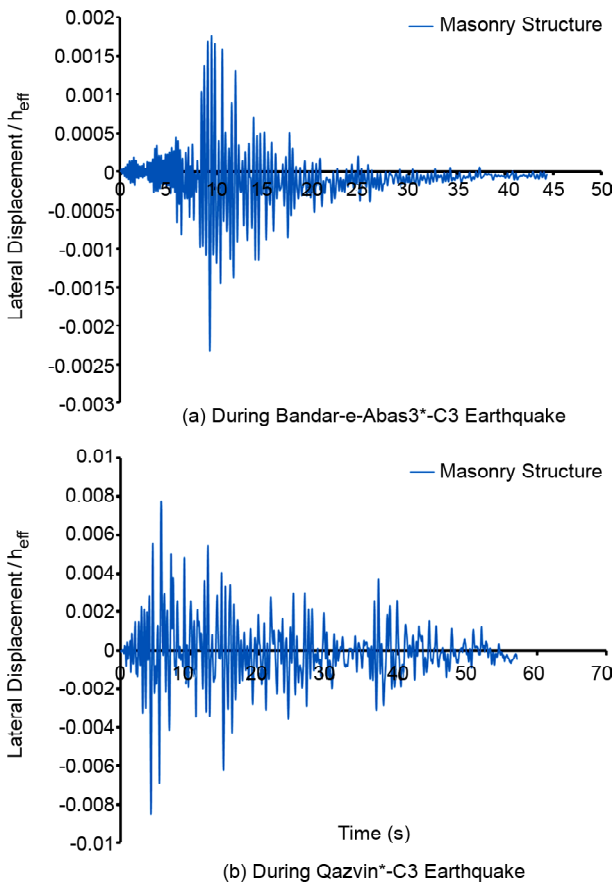
Figure 9. Time history drift response of building for north-south direction.

(a) During Bandar-e-Abas3*-C3 Earthquake
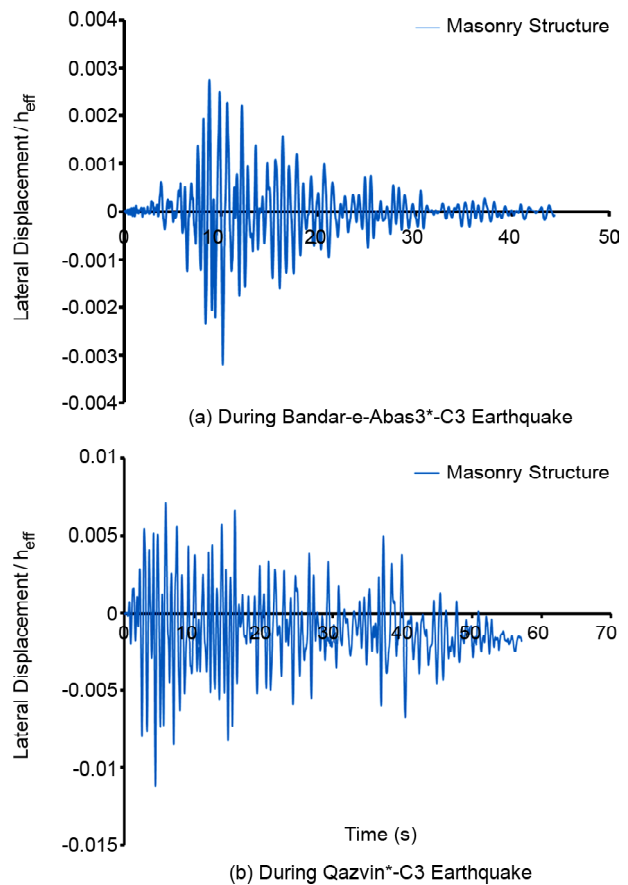
(b) During Qazvin*-C3 Earthquake

Figure 10. Time history drift response of building for east-west direction.

ground motion records, plastic deformations were occurred.

5. Conclusions

This study presents a practical approach for modeling of unreinforced masonry buildings. Some of the important findings of this study are as follows:

❖ This approach offers a good balance between simplicity and accuracy and provides easy and accessible understanding of the masonry building response.

❖ The nonlinear static and dynamic analyses have consistent results using the proposed approach.

❖ The seismic assessment of a case study as a representative building that selected based on a statistical study of Iranian masonry schools, shows this typical school would suffer structural damage in an earthquake having similar ground motion characteristics.

❖ In some ground motion records, plastic deformations were occurred in this typical school

building.

❖ "Expected shear strength" determine from test results, has the most effect on seismic assessment result of a masonry building.

❖ At most of masonry walls of these schools, governing mode was toe crushing; it is because of effect of axial loads.

❖ Results of seismic assessment of this typical school show that a large part of Iranian masonry schools need to retrofit.

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