The Vulnerability Assessment and Seismic Rehabilitation of the Historical Malek Zouzan Mosque

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SUMMARY:
Non-engineering masonry buildings illustrate immense unspecific and inappropriate seismic behavior. Since these buildings are utilized extensively in large and small cities, including the rural areas of Iran, hence, these types of structures are considered very important in relative to seismic assessments. Seismic vulnerability assessment methods of buildings and masonry walls comprise of the simple code method, complex method of Finite or Discrete Elements and applied Equivalent Frame Method. Due to the absence of an applicable and suitable method for the assessment of the masonry buildings, this research renders relatively simple method which is capable of being applied to all masonry buildings such as, residential, schools, historical and religious edifices. In this research efforts have been made to achieve a combination of the FEM and EFM methods and modifying and combining the FEMA306 and FEMA376 guidelines to survey and assess the seismic behavior of a historical structure. The results of which, comprised of the following: Determining the lateral seismic capacity and ultimate displacement and also to verify the collapse modes of the building. The advantage of the proposed method is the seismic assessment and strengthening design is performed successively in a certain model.

Keywords: Seismic Vulnerability, Historical Building, Equivalent Frame Method, Strengthening

1. INTRODUCTION

The functioning of non-engineering masonry building in past earthquake was not suitable. Due to the presence of an immense number of masonry buildings in Iran and other parts of the globe vulnerability seismic assessment is important. There were numerous researches performed to determine the capacity and mechanisms of in-plane failure modes which are necessary for vulnerability seismic assessments. A number of researchers have investigated on complex FE and Discrete Element Methods; and a group of researchers have investigated on more applicable methods such as, Equivalent Frame Method (Alemi et al. 2010 & 2006, Kappos et al. 2002).

In order to rehabilitate buildings against earthquakes there is a need for an assessment of seismic vulnerability and recognition of seismic failure modes. The more accurately this phase is conducted; a better seismic rehabilitation of a building is capable of being performed economically, both in cost and time.

The authors of this paper have proposed the Modified Equivalent Frame Method to assess relatively complicated historical walls (Alemi et al. 2010). The mathematical models for analyzing the wall are composed of linear finite elements and nonlinear equivalent frames. In this article, the mentioned method has been utilized for the assessment of a 3D historical building. Fig 1.1 illustrates the perspective view of the historical mosque.

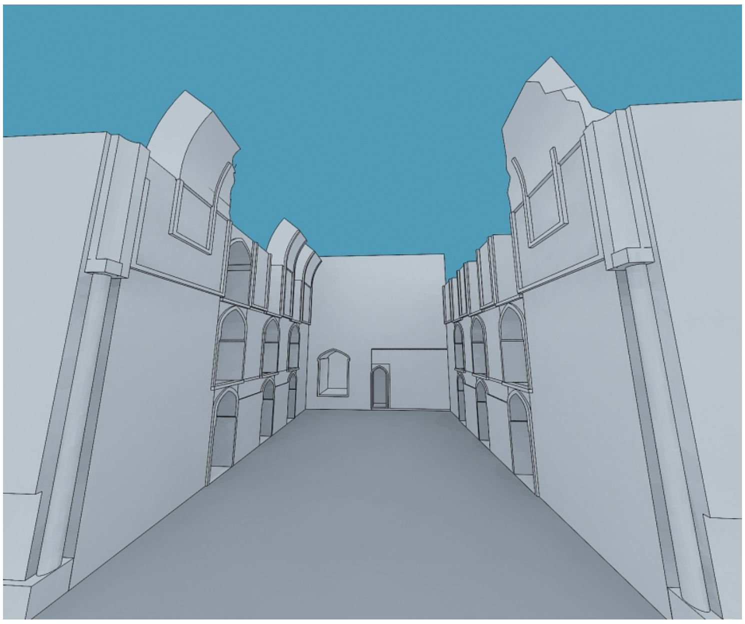


Figure 1.1 Perspective of the Historical Malek Zouzan Mosque

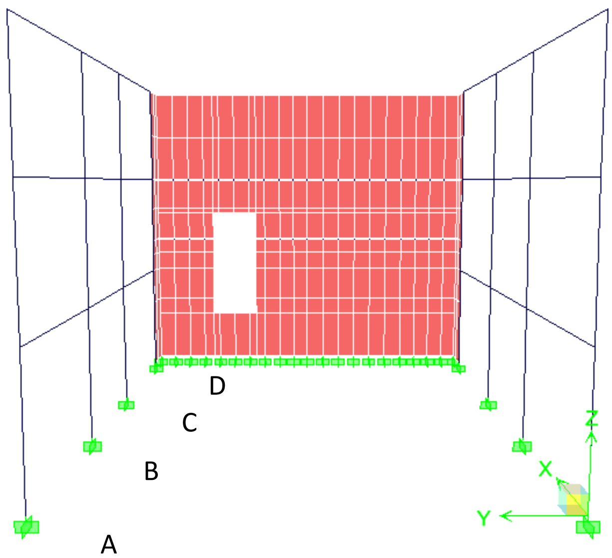


Figure 2.1 3D analytical model of the Malek Zouzan Mosque

2. MODELING

In this research in order to gain access to an applicable and relatively easy method in modeling masonry building, plain walls have been modeled by the combinations of the linear shell elements and nonlinear frame elements which connect the shells. The piers or walls with large openings are modeled by equivalent frames. Fig 2.1 shows such an analytical model.

In accordance with the FEMA 306 and FEMA 376 guidelines (FEMA-306 1998, FEMA-376 2000), the possibility of the occurrence of cracks due to the bed joint sliding, the rocking and the toe crushing modes are more probable and in the mid height of piers, the bed joint sliding and diagonal tension modes are probable.

The relations which could utilize in the non-linear modeling of the joints of piers are described as follows:

$$V_{bjs1}=0.375\ v_{te}A+0.5P_{CE}\ \ \&\ V_{bjs2}=0.5P_{CE} \tag{1.1}$$

$$V_r=0.9\ \alpha P_{CE}\ (L/H_{eff}\ ) \tag{1.2}$$

$$V_{tc}=\alpha P_L\ (L/H_{eff}\ )(1-f_a/0.7f'_m) \qquad , \frac{L}{H_{eff}} \geq 0.67 \tag{1.3}$$

$$V_{dt}=f'_{dt}A(L/H_{eff}\ )\sqrt{1+f_a/f'_{dt}} \qquad , 0.67 \leq \frac{L}{H_{eff}} \leq 1 \tag{1.4}$$

According to the above formulas, three kinds of the capacities of hinges at the two extremes of each pier could be compared and the minimum amount of them assigned to the joint:

$$Pier\ \ Mode\ \ (Top\ \ \&\ Bot\ \ Hinge):\min\{V_{bjs}\ \ , V_r\ \ , V_{tc}\} \tag{1.5}$$

And also for the joints in the mid-height of every pier:

$$Pier\ \ Mode\ \ (Middle\ \ Hinge):\min\{V_{dt}\ \ , V_{tc}\} \tag{1.6}$$

The parameters stated in relative to the above mentioned are:

V_{bjs1}	The bed joint siding capacity in relative to the mortar adhesiveness
V_{bjs2}	The bed joint sliding capacity without taking the adhesiveness into consideration
V_r	Rocking Capacity
V_{dt}	Diagonal Tension Capacity
V_{tc}	Toe Crushing Capacity
v_{te}	Shear strength of the mortar
A	Pier cross section
P	Existing vertical load of the pier ($P=P_{Dead}+P_{Live}$)
P_{CE}	Expected vertical load ($P_{CE}=1.1P$)
P_L	Vertical load ($P_L=0.9P$)
α	Constant coefficient which is 0.5 for the cantilever pier and 1 for the fix end pier
L	Height of pier
H_{eff}	Efficient height of the pier
f'_{dt}	Diagonal tension strength of the mortar (from in-situ test or from the formula no (1.7))

$$f'_{dt}=v_m=0.375\ v_{te}+0.5\frac{P_{CE}}{A} \tag{1.7}$$

f_a	Existing normal stress in the pier (from flat jack test)
f'_m	Compression strength of the masonry

f_{me} Efficient compression strength

In a pier where the toe crushing mode dominates, this mode alone is allotted to ¼ joint on each side of the pier and the remaining joints (in the width of the pier) are introduced by the next dominating mode.

Similarly, at the two extremes of the spandrels, one bending joint and in the center of the spandrels two shear joints have been conducted, which shall be described as follows:

$$M_r = \left(0.375\, v_{te} + 0.25 \frac{P_{CE}}{A_n}\right) \times \frac{b_w \times b_l}{2} \times \frac{d_{sp}^2}{6\, b_h} \tag{1.8}$$

$$V_{dt} = f'_{dt} \cdot A_n \left(\frac{L_{sp}}{d_{sp}}\right) = 0.375\, v_{te} \cdot A_n \left(\frac{L_{sp}}{d_{sp}}\right) \quad , 0.67 \le \frac{L_{sp}}{d_{sp}} \le 1 \tag{1.9}$$

Parameters utilized in the above mentioned formulas are:

M_r	Rocking of the two ends of beams
b_w	Width of one brick
b_l	Length of one brick
b_h	Thickness of a brick in addition to the horizontal layer of mortar
d_{sp}	Depth of spandrel in the location under consideration (end or mid of span)
L_{sp}	Length of span
A_n	Cross-section of the spandrel in the location under consideration (end or mid of span)

The non-linear behavioral curve of ductile and brittle joints such as Fig. 2.2 should be introduced. To have a larger safety margin it is convenient to overlook the DE branch of the curve in Fig 2.2(a).

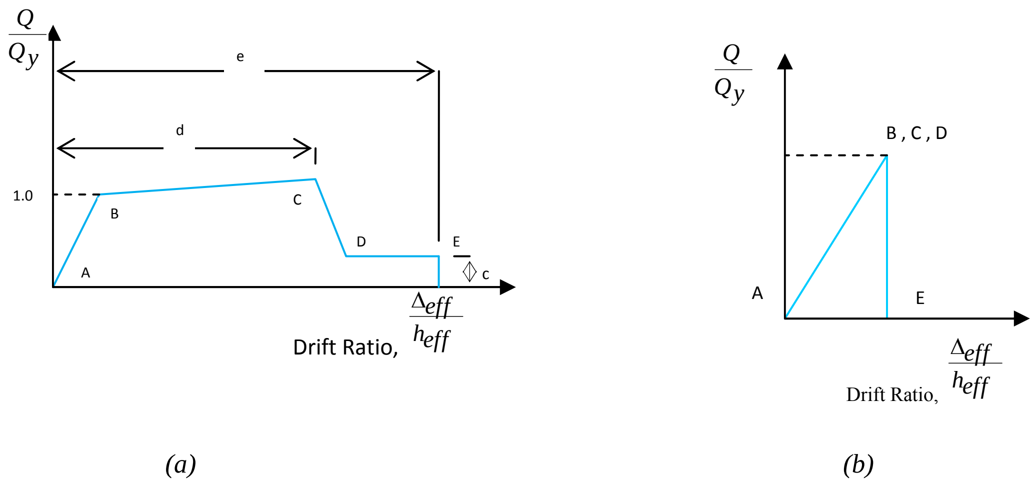


Figure 2.2. Force ratio - displacement ratio curve, (a) ductile, (b) brittle

3. ANALYSIS

The masonry building was 3D modeled and its natural periods were computed in two major directions (Table 3.1).

Table 3.1. Modal Periods and Frequencies (Tx≈0.312 sec, Ty≈0.1996 sec)

Mode	Period	Frequency	Circ. Freq.
	Sec	Cycle/sec	rad/sec
1	0.31202	3.20497	20.13742
2	0.29845	3.35069	21.05301
3	0.29295	3.41351	21.44769
4	0.28058	3.56405	22.39361
5	0.20772	4.81418	30.24838
6	0.19964	5.00912	31.47325

The fundamental period of the structure has calculated by a linear analysis and it is used to determine the target displacement for the push over analysis. In order to compute the target displacement, the following formula is utilized:

$$\delta = C_0 . C_1 . C_2 . C_3 . S_a . \frac{T_e^2}{4\pi^2} . g \tag{3.1}$$

The coefficients of C_0, C_1, C_2, C_3 and S_α have been determined according to the Iranian guideline for the seismic rehabilitation of the present structures (Publication No. 360), as follows:

X direction: C0=1.2, C1=1.323>1, C2=1.39, C3=1 (3.2)

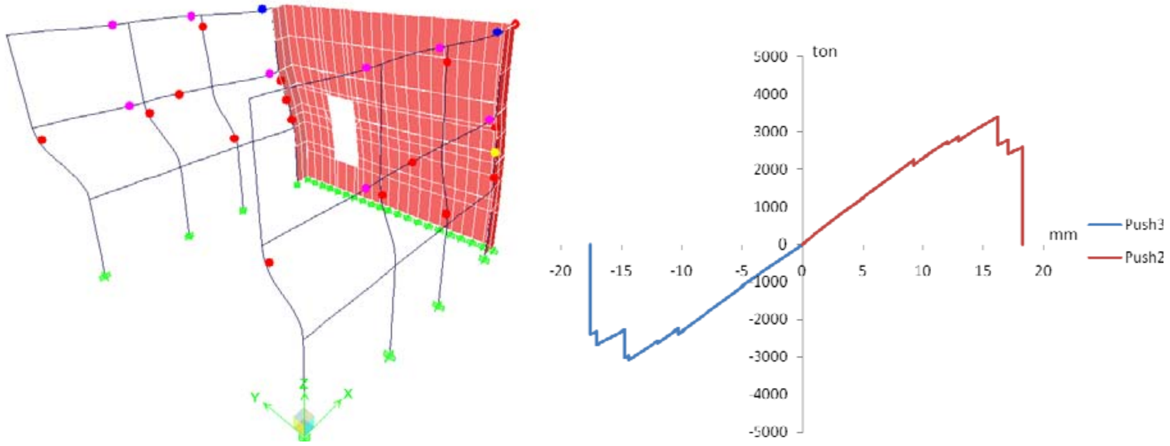
Y direction: C0=1.2, C1=1.41>1, C2=1.45, C3=1 (3.3)

$$S_a = 0.36 \times 2.75 \times 1 = 0.99 \tag{3.4}$$

$$\delta = 1.2\times1.323\times1.39\times1\times(0.30\times2.75\times1.4)\times(0.312)^\wedge2/(4P^\wedge2)\times9.81\times100=6.16\text{cm} \tag{3.5}$$

$$\delta = 1.2\times1.41\times1.45\times1\times(0.30\times2.75\times1.4)\times(0.1996)^\wedge2/(4P^\wedge2)\times9.81\times100=2.82\text{cm} \tag{3.6}$$

Fig. 3.1(a) and (b) shows the nonlinear joints and Fig. 3.2(a) and (b) illustrate the force-displacement curves of the structure under push over analysis. In this analysis the dominating collapse modes, seismic capacities and an approximation of the maximum displacement of the structure can be estimated. In Tables 3.2 and 3.3 the position of nonlinear joints and types of the collapse modes are presented.



(a) Nonlinear joints in frame elements (b) Pushover curves

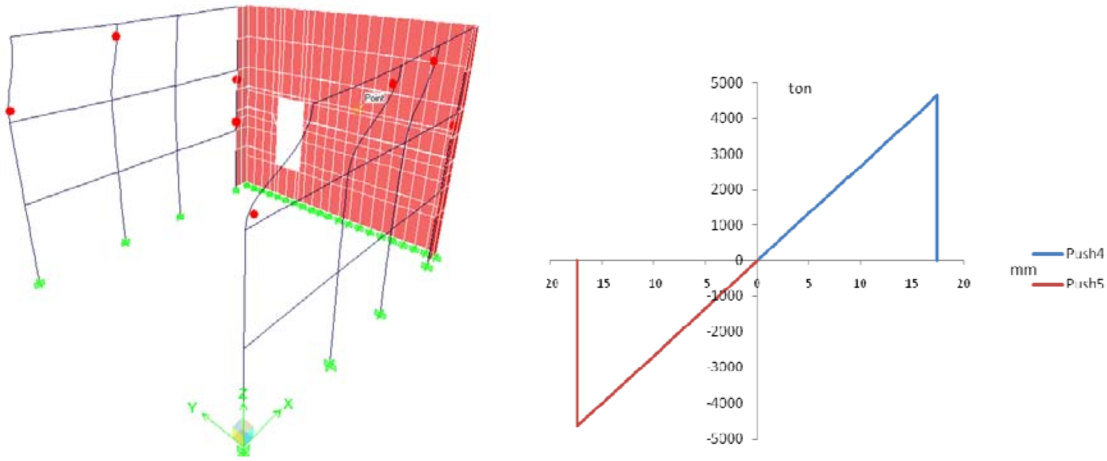
Fig 3.1. Results of the pushover analysis in X-direction (Strength capacity=2930 tons, Maximum displacement=18.2mm)

Table 3.2. The position of nonlinear joints and the types of the collapse modes in X-direction

N.	Station	Event	Mode	Color
1	B.CD-3-R	Crack	Bed joint sliding	
2	P.D-2-T	Collapse	Toe crushing	
3	B.AB-3-R	Crack	Bed joint sliding	
3	B.BC-2-M	Collapse	Diagonal tension	
4	B.BC-3-R	Crack	Bed joint sliding	
4	P.D-2-B	Collapse	Toe crushing	
5	P.A-2-T	Collapse	Toe crushing	
6	P.B-2-T	Collapse	Toe crushing	
7	P.C-2-B	Collapse	Toe crushing	
8	P.C-3-T	Collapse	Toe crushing	
8	B.CD-2-R	Crack	Bed joint sliding	
9	P.D-2-M	Collapse	Bed joint sliding	

Table 3.3. The position of nonlinear joints and the types of the collapse modes in Y-direction

N.	Station	Event	Mode	Color
1	P.D-2-T	Collapse	Toe crushing	
2	P.D-2-B	Collapse	Toe crushing	
3	P.A-3-B	Collapse	Toe crushing	
3	P.B-3-T	Collapse	Toe crushing	
4	P.C-3-T	Collapse	Toe crushing	



(a) Nonlinear joints in frame elements (b) Pushover curves

Figure 3.2. Results of the pushover analysis in Y-direction (Strength capacity=4640 tons, Maximum displacement=17.4mm)

4. SEISMIC REHABILITATION

The utilized method for preparing seismic rehabilitation plan is to eliminate the brittle modes and to make the ductile modes dominant by increasing the compression stress and the various strengths of the walls by using the post tension cables. In order to determine the required increment of the tensions of the cables in the piers, the capacity of the joints are modified according to the required increase of the wall strength to eliminate the brittle modes, then this capacity analysis is repeated until the target performance is achieved.

In the successive analysis, the collapse modes are specified and the brittle modes are eliminated by the above mentioned operation.

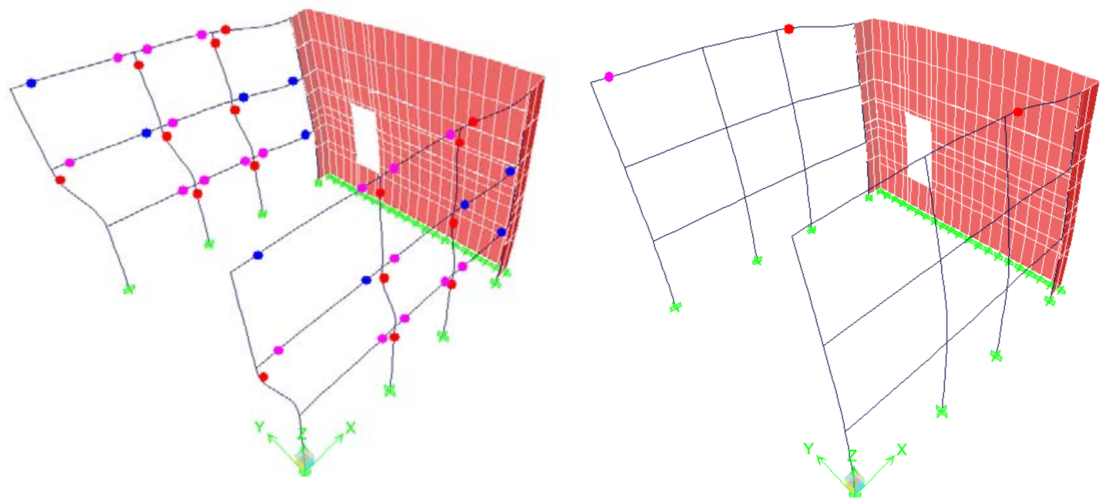
The final results in the last stage of the analysis to attain the performance level under consideration are as follows:

- Increase in the corner piers compression stress equal to 10.5 kg/cm2(D axis)
- Increase in the corner piers compression stress equal to 3.34 kg/cm2(A axis)
- There was no requirement to increase the compression stress in piers of the B and C axis

In Fig. 4.1 and Table 4.1 and 4.2 the results of the pushover analysis of the rehabilitated structure are shown. In Fig. 4.2 and Table 4.3 the comparison of the results of the pushover analysis between the rehabilitated and original structure are shown.

Table 4.1. The position of nonlinear joints and the types of the collapse modes in Y-direction in rehabilitated structure

N.	**Station**	**Event**	**Mode**	**Show**
1	B.AB-3-L	Crack	Bed joint sliding	
2	B.CD-3-L	Collapse	Bed joint sliding	

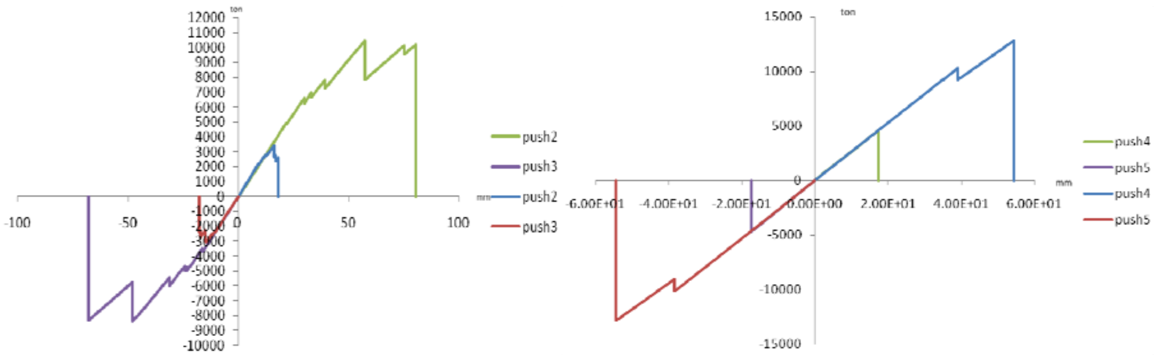


(a) X-direction (Strength capacity=7831 tons, Maximum displacement=61.6mm)
(b) Y-direction (Strength capacity=7424 tons Maximum displacement=28.2mm)

Figure 4.1. Results of the pushover analysis of the rehabilitated structure

Table 4.2. The position of nonlinear joints and the types of the collapse modes in X-direction in rehabilitated structure

N.	Station	Event	Mode	Color
1	P.A-2-T	Collapse	Toe crushing	
2	P.B-1-T	Collapse	Toe crushing	
3	P.B-2-T	Collapse	Toe crushing	
4	P.B-3-T	Collapse	Toe crushing	
5	P.C-1-T	Collapse	Toe crushing	
6	P.C-2-T	Collapse	Toe crushing	
7	P.C-3-T	Collapse	Toe crushing	
8	B.AB-1-R	Crack	Bed joint sliding	
9	B.AB-2-R	Crack	Bed joint sliding	
10	B.AB-2-L	Crack	Bed joint sliding	
11	B.AB-3-R	Crack	Bed joint sliding	
12	B.AB-3-L	Crack	Bed joint sliding	
13	B.BC-1-R	Crack	Bed joint sliding	
14	B.BC-1-L	Crack	Bed joint sliding	
15	B.BC-2-R	Crack	Bed joint sliding	
16	B.BC-2-L	Crack	Bed joint sliding	
17	B.BC-3-R	Crack	Bed joint sliding	
18	B.BC-3-L	Crack	Bed joint sliding	
19	B.CD-1-R	Crack	Bed joint sliding	
20	B.CD-1-L	Crack	Bed joint sliding	
21	B.CD-2-R	Crack	Bed joint sliding	
22	B.CD-2-L	Crack	Bed joint sliding	
23	B.CD-3-L	Collapse	Bed joint sliding	



(a) X-direction (b) Y-direction

Figure 4.2. Comparison of the pushover curves before and after strengthening

Table 4.3. Comparison of the capacities before and after strengthening

Capacity	After rehabilitation		Before rehabilitation	
	y	x	y	x
Strength(tons)	7424	7831	4640	2930
Displacement(mm)	28.2	61.6	17.4	18.2

5. CONCLUSION

The original FE model of the building which has developed according to the Modified Equivalent Frame Method, can be utilized to determine the effects of the incremental strengthening of the masonry walls so it is possible to determine the level of the required strengthening of each part of the building separately.

According to the above mentioned applications of the Modified Equivalent Frame Method, it is proposed to utilize for rapid assessment of the masonry buildings.

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