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The Vulnerability Assessment of Historical Masonry Buildings Against Earthquakes by the Modified Equivalent Frame Method

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Abstract Insufficient ductility and lack of integrity have caused extensive damages and undesirable responses in masonry buildings in past earthquakes. These types of structures are distributed all over Iran and the vulnerability assessment of such structure is one of the main concerns between structural engineers. Developing an advance finite element model to simulate the behaviour of a masonry structure is a very complicated procedure which may not be practical and applicable in most cases, so many attempts have been performed to find simple methods for analysing these structures. The Equivalent Frame Method is one of the well-known methods which is rather simple and could be used for vulnerability assessment of some historical structures like Iranian caravanserais. The architectural configurations of Iranian caravanserais were regular and symmetric. When the arrangement of walls and openings are modular it is convenient to model the piers and spandrels as a frame type columns and beams. The simplicity of the conventional Equivalent Frame Method sometime leads to an inaccuracy of the assessment, therefore some modification for the upgrading of this method has been proposed. Using the shell elements beside the frame elements in a modelling procedure may eliminate some disadvantages of the conventional Equivalent Frame Method.

In this study a modification has been proposed to improve the upgraded Equivalent Frame Method ability in predicting the failure modes of the masonry walls such as, toe crashing. The proposed modification has been used in the static nonlinear analysing of a masonry structure by the Equivalent Frame Method. The given results have acceptable reconciliation with practical reality and it seems to be useful, especially in a rapid assessments.

Keywords: Masonry wall, equivalent frame method, vulnerability assessment,performance level

Introduction

In this research, in order to gain an access to a practical method, which is relatively easy in modeling masonry structures, a combination of two methods, the Finite Element Method and the Equivalent Frame Method (Giordano et al. 2002, Kappos et al. 2002) have been utilized. The former has been used in modeling the piers as macro elements and the latter in for modeling the non-linear joint between macro elements. To describe the specifications of the non-linear joint for the push over analysis, the FEMA 306 and FEMA 376 guidelines have been combined and modified to be able to simulate all of the common damage modes in the piers and spandrels in an analyzing procedure.

Modeling Methodology

In Fig. 1, a two storey masonry structure and a sample pattern of the location of the joints between the macro elements of the masonry parts has been shown. Nonlinear behavior of each joint depends on its location, and dominates by shear or normal stresses. At each joint, the capacities against all of the probable damage modes could be compared and the critical one must be utilized for the modeling.

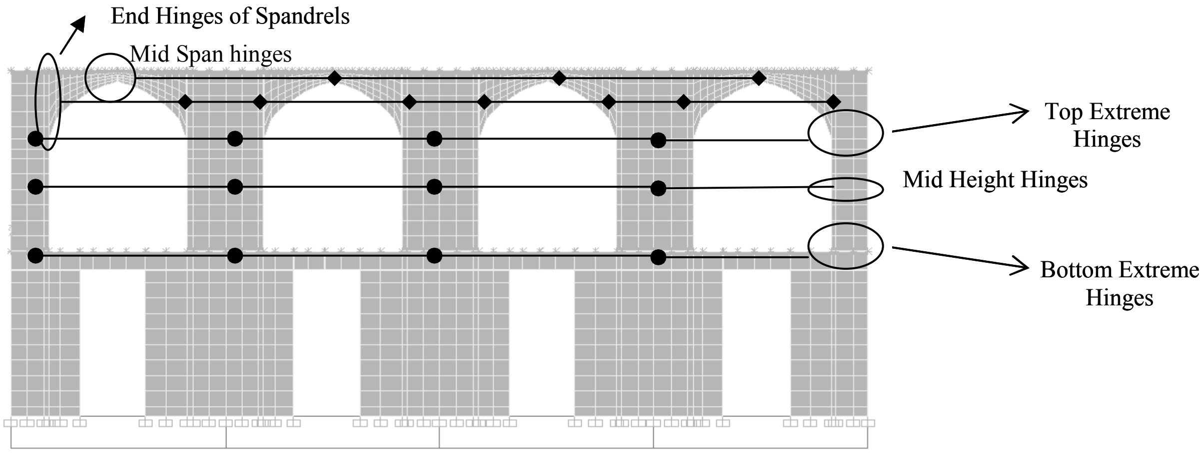


Fig. 1: Typical modeling of the structure and location of joints in every storey

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:

$$V_{bjs1}=0.375\ v_{te}A+0.5P_{CE}\ \ \&\ V_{bjs2}=0.5P_{CE} \tag{1}$$

$$V_r=0.9\ \alpha P_{CE}\,(L/H_{eff}) \tag{2}$$

$$V_{tc}=\alpha P_L\,(L/H_{eff})(1-f_a/0.7f'_m)\quad,\qquad \frac{L}{H_{eff}}\geq 0.67 \tag{3}$$

$$V_{dt}=f'_{dt}A\,(L/H_{eff})\sqrt{1+f_a/f'_{dt}}\quad,\qquad 0.67\leq\frac{L}{H_{eff}}\leq 1 \tag{4}$$

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:

$$Pier\ Mode\ (Top\ \&\ Bot\ Hinge):\min\{V_{bjs}\ ,V_r\ ,V_{tc}\} \tag{5}$$

and also for the joints in the mid-height of every piers:

$$Pier\ Mode\ (Middle\ Hinge):\min\{V_{dt}\ ,V_{tc}\} \tag{6}$$

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 (7))

$$f'_{dt}=v_m=0.375\,v_{te}+0.5\frac{P_{CE}}{A} \tag{7}$$

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:

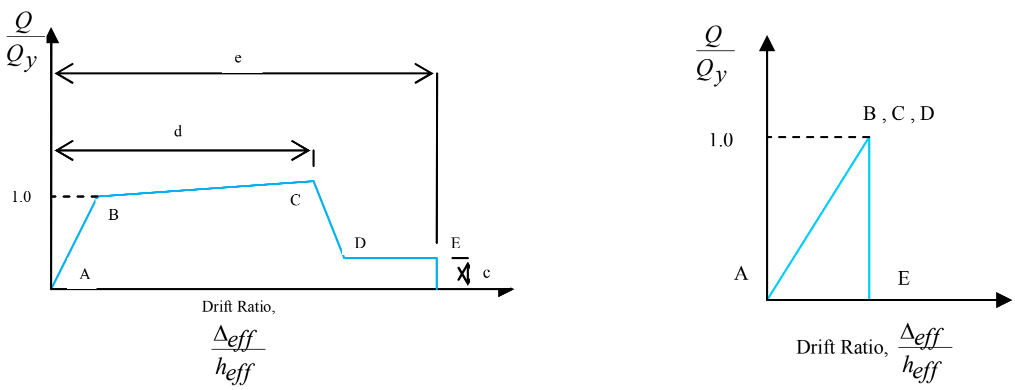
$$M_r=\left(0.375\,v_{te}+0.25\frac{P_{CE}}{A_n}\right)\times\frac{b_w\times b_l}{2}\times\frac{d_{sp}^2}{6\,b_h} \tag{8}$$

$$V_{dt}=f'_{dt}\,A_n\left(\frac{L_{sp}}{d_{sp}}\right)=0.375\,v_{te}\,A_n\left(\frac{L_{sp}}{d_{sp}}\right)\quad ,\quad 0.67\le\frac{L_{sp}}{d_{sp}}\le 1 \tag{9}$$

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 should be introduced. To have a larger safety margin it is convenient to overlook the DE branch of the curve in Fig. 2(a).



(a) *(b)*

Figure 2: Force ratio - displacement ratio curve, (a) ductile, (b) brittle

Case Study

To present the capability of the mentioned methodology, the vulnerability of a typical historical masonry wall has been studied. The wall consisted of some openings, piers and spandrels which it has been shown in the Fig. 1. After the vulnerability assessment, a rehabilitation method has been chosen, designed and introduced to the model. The differences of the behavior of wall prior and after rehabilitation have been studied to find the optimal rehabilitation result.

Modeling and Analyzing The under consideration wall is a part of a masonry structure which has been constructed about 40 years ago in a high seismic zone of Iran. The specification of the bricks and mortar have been determined through the in-situ tests and used for the analysis:

$v_{te} = 7\ kg/cm^2$

$f'_m = \frac{f_{me}}{1.6} = \frac{95}{1.6} \approx 60\ kg/cm^2$

$b_h = 5.5 + 2\ cm$

$b_l = 21\ cm$

$b_w = 11\ cm$

$T \approx 0.243$ sec

T is the fundamental period of the structure which has been computed by a linear analysis and it has been used to define the target displacement for the push over analysis. In order to compute the target displacement, the following formula is utilized:

$$\delta = C_0 . C_1 . C_2 . C_3 . S_a . \frac{T_e^2}{4\pi^2} . g \qquad (10)$$

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:

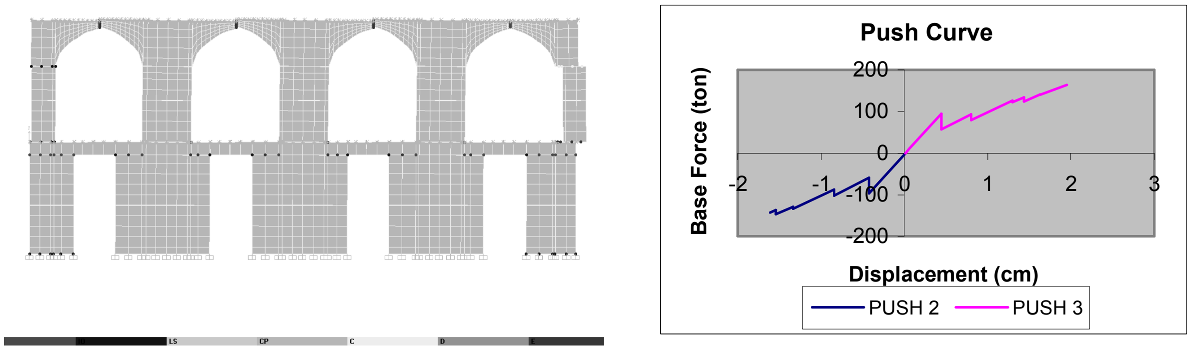
$C_3 = 1$ و $C_2 = 1.43$ ، $C_1 = 1.38 > 1$ ، $C_0 = 1.2$

$S_\alpha = ABI$

$S_a = 0.36 \times 2.75 \times 1.4 = 1.386$

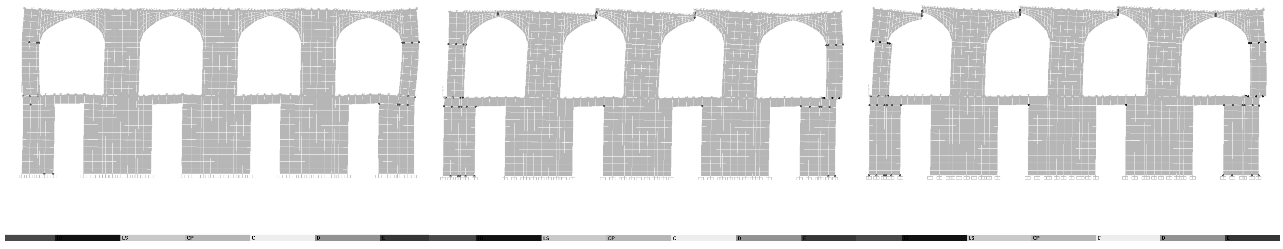
$$\delta = 1.2 \times 1.38 \times 1.43 \times 1 \times (0.36 \times 2.75 \times 1.4) \times \frac{(0.243)^2}{4\pi^2} \times 9.81 \approx 0.0482\ m \approx 4.82\ cm$$

Fig. 3 illustrates the results of a push over analysis by using the SAP 2000 Software based on the above data. Fig. 3(a) shows the yielded joints and Fig. 3(b) shows the force-displacement curve of the wall under push over analysis. In this analysis, as observed in Fig. 3(b), the model is afflicted to destruction upon attaining a capacity of 150 tons. This demonstrates the vulnerability of the wall under the performance level under consideration. In order to obtain an agreeable performance level the strengthening of the wall is required. In Fig. 4 the location of the initial cracks and destructive phases of the walls have been demonstrated



(a) (b)

Figure 3: push over analysis result, (a) Creation of a joint , (b) The two way push curve



(a) *(b)* *(c)*

Figure 4: Destruction development phases, (a) phase 1, (b) phase 2, (c) phase 3

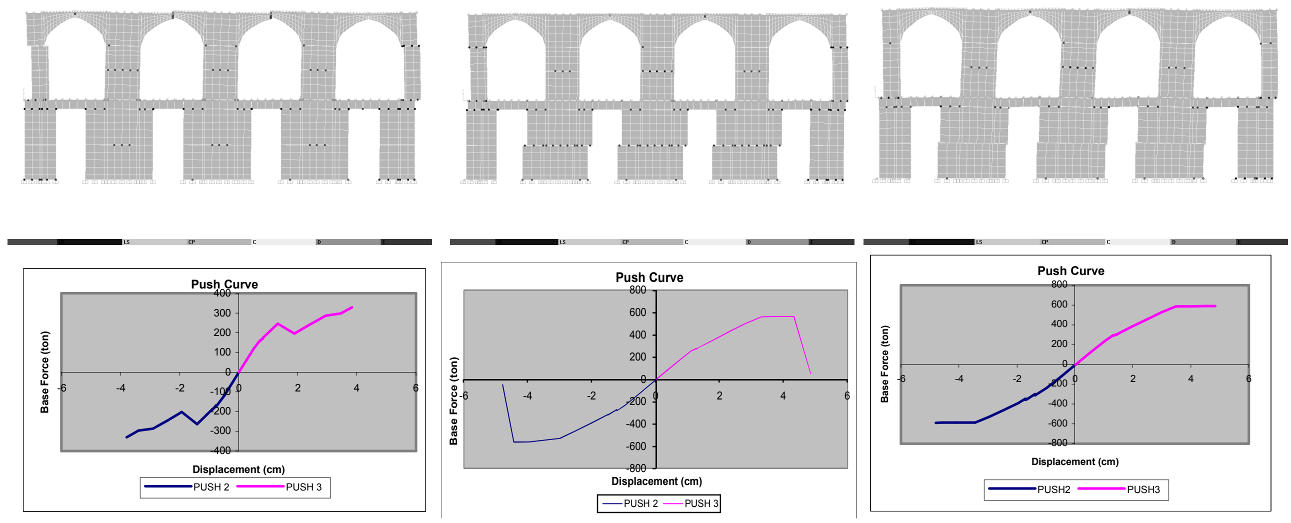
Seismic Rehabilitation Generally, one of the most efficient methods for seismic rehabilitation of such a masonry structures is to exclude the brittle modes and to make the ductile mode dominate by increasing the normal compression stress in piers and increasing the capacities of the piers (Alemi and Nateghi-Alahi 2006). So, inducing the normal stresses by utilizing post tensioned cables in the piers is the chosen method for the next assessments and analyzes.

In this case, by closing the subordinate failure modes, (for example by increasing the strength of the mid joints of spandrels) the survey and design of rehabilitation of the main failure modes have been done. It is important to mention that the local failure modes could be eliminated by some limited intervention like strengthening by using FRP or reinforcement bars.

To reach to the acceptable performance level, a group of analysis has been performed. At the end of each analysis, the demand in increasing the normal stress in the piers for modification of the capacities and dominated failure modes were determined. Increase in the vertical load in a direct way in formula no. 1 to 4 intervening incremental aspect. By reanalyzing, it would be possible to step by step become closer to the desirable performance level and find the required post tensioning stresses in each pier.

In the rehabilitation implementation according to the above mentioned method, it is observed that in some of the piers, the brittle mode of failure is transformed into the ductile nature and causes an increase in the pier ductility.

Fig. 5 illustrate the several phases and increment of the vertical tension in the columns.



(a) *(b)* *(c)*

Figure 5: push over analysis results in some phases of rehabilitation, (a) Third phase, (b) Sixth Phase, (c) Twelfth phase

Comparison of Results

In the rehabilitation of the above mentioned wall, by enforcing a pressure to the amount of 400 N/m2 to the middle piers and 800 N/m2 to the side piers and strengthening some of the weak points which have been previously mentioned, the piers have gained their suitable performance level and illustrate their suitable ductility. (Fig. 6 and Table 1)

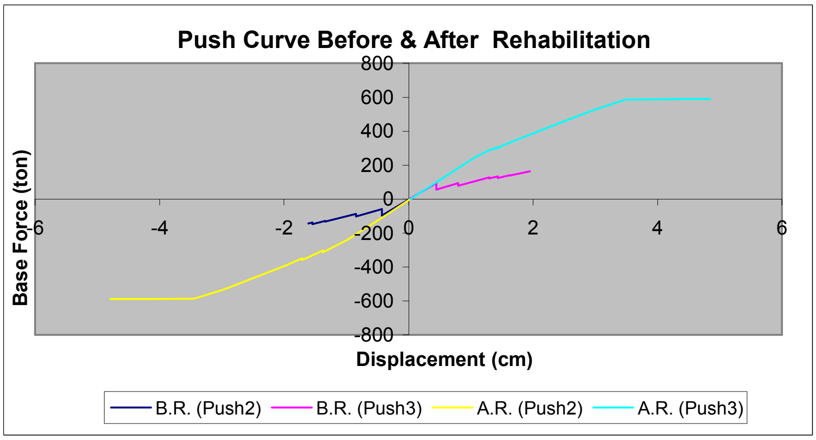


Fig. 6: Comparison of the push curve of the wall prior and after rehabilitation

Table 1: Comparison of Capacity and Displacement of the wall prior and after rehabilitation

	Before	*After*
Lateral force Capacity	164 ton	590 ton
Displacement capacity	1.95 cm	4.85 cm

Summery

A methodology for modeling and analyzing the masonry walls has been reviewed and the capability of the method for vulnerability assessment of a relatively complicated masonry wall presented. The case study shows that the combination of two methods, the Finite Element Method and the Equivalent Frame Method is a simple and efficient method and helps for rapid conclusions.

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