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Performance level of masonry structures.

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ABSTRACT

Focusing on the importance of resistive structure designing against dynamic loads such as earthquake load and in capability of the designing methods which are based on the force to predict the nonlinear behavior of member affected by nonlinear virtue of materials. In recent years the tendency of engineers and designers to use methods of designing based on displacement and behavior (designing according to performance) has been increased.

Because of being economic, and easy to accomplish and accessibility of the building material .nowadays we see a lot of masonry structures in villages and towns. In other hand Iran's location on the potential earthquake belt of alpine show the importance of studying about seismic behavior and vulnerability in this kind of structures.

Various factors such as environmental, economical, social and cultural condition and available materials caused the importance of various kinds of masonry structures. In this research in order to review the seismic vulnerability and performance level various samples of these structures are studied.

By applying a suitable model which is designed by computer software under pressure of dynamic load In addition the analytical result of implicated model will be compared with laboratory findings.

Assuming satisfactory used model, standard rights of seismic retrofitting about the number of analyzed structures has been evaluated by this model.

Key words: performance level, masonry structures, Seismic Vulnerability , accelerator suitable with geology conditions

INTRODUCTION

Studies on masonry buildings after earthquake shows that fragility of materials and non continuity of these buildings are main factor of destruction at the time of earthquake, There are similar damages occurred by earthquake on these structures Regarding to kind of construction of these structures. Some of these damages are horizontal cracks at the joint of ceiling and walls, indicated in Fig1. These cracks are caused due to unsuitability of connection between ceiling and walls .Also disconnection between walls at the joints and out surface collapse has been seen in most of masonry buildings at the time of earthquake Fig2, 3. As time passes along with research and experiments, using horizontal and vertical ties has been suggested to solve the problem. On the other hand, inertia force caused by earthquake on the building is divided on the structure elements in a way that a part by widthwise walls, a part by longitudinal walls and a part by ceiling are transferred. The proportion transferred from widthwise walls to ceiling and foundation and from ceiling to longitudinal walls. In fact, this mechanism defines the importance of longitudinal walls in transferring force caused by earthquake and the damage and destruction of them can have a noticeable effect on vulnerability and destruction of the whole structure. [1]

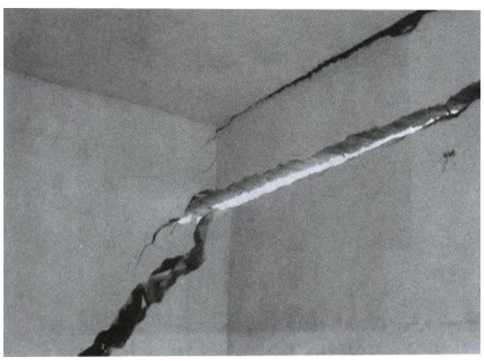

FIGURE 1. Deep cracks between ceiling and walls

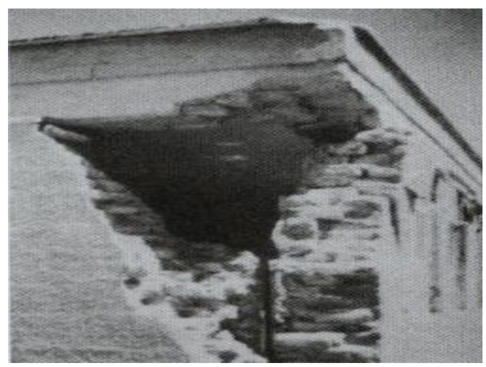

FIGURE 2. Discretion orthogonal walls

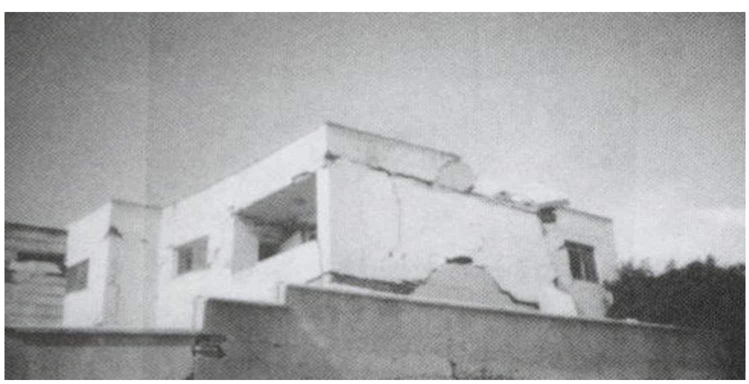

FIGURE 3. Out surface collapse of wall

In this study, destruction analysis method is introduced for confined brick wall and the effect of some parameters such as span length, numbers of span and number of floors on the destruction caused by earthquake. The common method to model of confined brick wall is taking equivalent diagonal member in to consideration. Park & Ang damage index and IDARC software has been used to analyze destruction. [2, 3]

MASONRY BRICK WALL

Compressive stress-strain diagram of masonry brick constructional material is illustrated in fig4. This diagram is considered as a parabolic function till the maximum stress of f_m' , then with the increase of strain the value of stress is decreasing linearly and after that is a fixed value. Assumed model for coverage of strength of brick panel is illustrated in fig 5 that this coverage model shows compressive behavior.

Cyclic force-displacement diagram for compressive situation is shown in fig 6. Resulted formulas for coverage are presented according to studies done by Sanee nezhad and Houbez. [4]

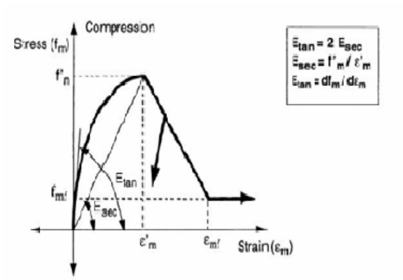

FIGURE 4. Stress-strain behavior of brick materials

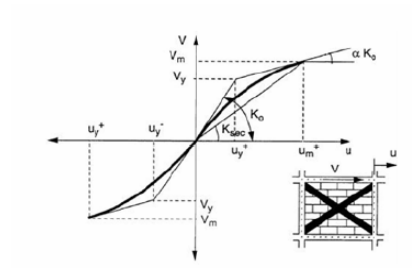

FIGURE 5 brick panel strength coverage

With taking a confined brick wall in fig 7 in to consideration, maximum lateral force V_m and the relevant displacement U_m are as following; [4]

$$V_m \leq A_d f_m' \cos\theta \leq \frac{vtl'}{(1-0.45\tan\theta')\cos\theta} \leq \frac{0.83(Mpa)tl'}{\cos\theta} \quad (1)$$

$$U_m = \frac{\varepsilon_m' L_d}{\cos\theta} \quad (2)$$

Where t is the thickness of wall, f_m' is prism strength of masonry materials; ε_m' is relevant strain, V is basic shear resistance or tenacity of masonry, A_d , L_d are area and equivalent length of diagonal member respectively . [5]

$$A_d = (1-\alpha_c)\alpha_c t h' \frac{\sigma_c}{f_c} + \alpha_b t l' \frac{\tau_b}{f_c} \le \frac{0.5h' \frac{f_a}{f_c}}{\cos\theta} \tag{3}$$

$$L_d = \sqrt{(1+\alpha_c)^2 h'^2 + L'^2} \tag{4}$$

The values of α_c ،α_b ،τ_b ،f_a و f_c are related to geometry and the characteristics of confined material and wall (infill frame). Allowable stress f_a is resulted from the Equations following;

$$f_c = 0.6 \times \phi \times f_m{}' \qquad f_a = f_c\left[1 - \left(\frac{L_{eef}}{40\ t}\right)\right], \qquad \phi = 0.65 \tag{5}$$

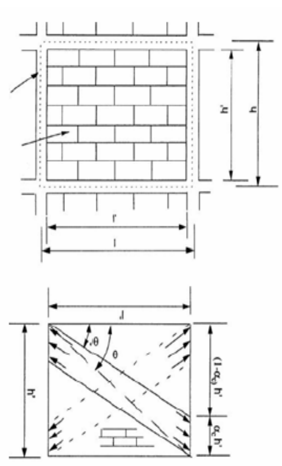

FIGURE 6. brick panel and equivalent compressive member

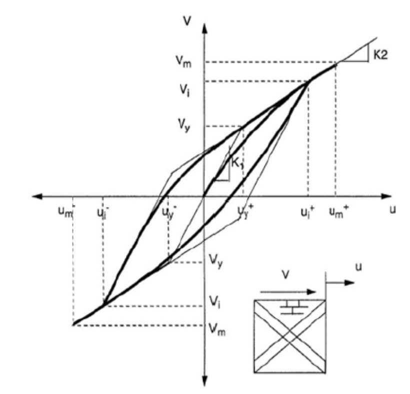

FIGURE 7 VAN-Bouk model for Cyclic response of brick panel

Uniform contact stresses at the time of fracture at the joint of vertical ties with wall σ_{c0} and horizontal ties with brick wall σ_{bo} based on Tereska hexagonal gauges yielding is as following ;

$$\sigma_{co} = \frac{f_c}{\sqrt{1+3\mu_f{}^2 r^4}} \qquad \sigma_{b0} = \frac{f_c}{\sqrt{1+3\mu_f{}^2}} \tag{6}$$

Where r is height of wall to length of wall ratio and μ_f is friction factor of wall with ties .In one dimensional force-displacement diagram, maximum force of V_m And the relevant displacement U_m , initial stiffness K_0 and. stiffness of ultra yielding to initial stiffness ratio α are considered. The initial stiffness K_0 can be estimated as following;

$$k_0 = \frac{V_y}{U_y} \tag{7}$$

Lateral yielding force and the relevant displacement in brick panel are as following;

$$V_y = \frac{V_m - \alpha K_0 U_m}{1-\alpha} \tag{8}$$

$$U_Y = \frac{V_m - \alpha K_0 U_m}{K_0(1-\alpha)} \tag{9}$$

For α the value 0.1 is suggested.

PERFORMANCE LEVEL AND ACCEPTANC CRITERIA

Different codes for performance based design have similar interpretations. The goal of seismic design of buildings is defined for different kinds separately and the selection of performance of structure is on the employer decision so the designer should take it for safety in to consideration.

Performance level is showing some boundary conditions where the acceptable value and amount of damage of a building because of an earthquake is defined. These boundary conditions are described by physical damage in the structure and the life danger for residents inside the building and the amount of serviceability of structure after the earthquake. A building consists of complicated elements and members which many of these elements have an independent performance than other elements, so for designing level, different kinds of elements performance should be considered.

Elements of a structure have 4 main performance levels as following;

1. Immediat occupancy performance level
2. Life safety performance level
3. Collapese prevention performance level
4. Not considered performance level

to study a structure performance ; result of analysis must be compared with some boundary values of performance where these boundary values are categorized in 2 parts of general acceptable criteria of a building and acceptable criteria of elements. And if the result of each analysis be more than the acceptable criteria then performance level is not accepted.

The general acceptance criteria of building as following;

Resistance against of lateral load: According to ATC 40 the lateral resistance of structure at performance point should not decrease more than 20 percent of ultimate resistance [7] and according to FEMA-273 base shear at performance point should not be less than 80 percent of base shear at limit yielding. [8]

Lateral displacement; Lateral displacement is controlled by relative lateral displacement conception. So the allowable limits for relative lateral displacement and inelastic lateral displacement are as table (1). [7]

TABEL 1. allowable relative displacements in different performance levels

	Immediate occupancy	**Damage control**	**Life safety**	**Collapse prevention**
relative lateral displacement	0.01	0.01-0.02	0.02	0.33 Vi/Pi
inelastic relative lateral displacement	0.005	0.005-0.015	unlimited	unlimited

CONSIDERD WALLS

Walls of a building, with 3 and 5 meters span and the height of 3 meters in one floor and 2 floor buildings have been analyzed by nonlinear dynamic analysis.

In theses walls section and bars of ties are considered according to the Standard code 2800 of Iran. It is noted that load bearing surface of wall is 4 meters. Also, the used records related to Mexicosity, Northridge, Bam, Naghan, and Tabas earthquake are with maximum accelerator of 0.3g. To ease the work , tied walls have abbreviated like nS_mP_L where n is number of floors , m number of span , L is length of span and S,P are abbreviation form of story and panel respectively.

EFFECT OF SPAN LENGHT

To study the effect of span length, walls with the span of 3, 4, and 5 meters for mentioned earthquakes have been analyzed by nonlinear dynamic analysis and the results are based on total relative displacement of roof as following ;(table2)

TABEL 2. total relative displacement of roof

	Bam l	**Bam T**	**Naghan**	**Tabas**	**Northridge**	**Mexicosity**
1S-1P-3	0.0036	0.0035	0.0033	0.0054	0.0043	0.0038
1S-1P-4	0.0034	0.0033	0.0029	0.0042	0.0039	0.0035
1S-1P-5	0.0031	0.003	0.0024	0.0039	0.0034	0.0031

Results shows that in all earthquakes along with the increased of span length, total relative displacement of roof is decreasing. Increase of span length increases the stiffness of wall which is a factor to decrease the displacement.

EFFECT OF NUMBERS OF SPANS AND FLOORS

To study the effect of number of spans and number of floors, tied walls of 3,4,5 meters of one and two spans in one and two floors have been analyzed by nonlinear dynamic analysis. Results are as following ;(table3)

TABEL 2. total relative displacement of roof

	Bam L	Bam T	Naghan	Tabas	Northridge	Mexicosity
1S-2P-3	0.0033	0.0028	0.0025	0.0041	0.0038	0.0035
1S-2P-4	0.003	0.0025	0.0022	0.0037	0.0034	0.0033
1S-2P-5	0.0029	0.0022	0.0019	0.0034	0.0031	0.0030
2S-1P-3	0.0240	0.0215	0.0216	0.0260	0.0261	0.0153
2S-1P-4	0.0189	0.0185	0.0171	0.0227	0.0241	0.0123
2S-1P-5	0.0173	0.0145	0.0127	0.0189	0.0225	0.0103
2S-2P-3	0.0230	0.0202	0.0187	0.0242	0.0205	0.0130
2S-2P-4	0.0182	0.0188	0.0145	0.0189	0.0193	0.0110
2S-2P-5	0.0158	0.0154	0.0112	0.0163	0.0148	0.0099

Results shows that increase in number of spans causes decrease in total relative displacement which is influenced by increase of stiffness but increase in number of floors will cause increase in total displacement of roof.

RESULT

After study the results of analysis , we can say that in 1S-1P-3,1S-1P-4,1S-1P-5,1S-2P-3,1S-2P-4 and 1S-2P-5 the total relative displacement of roof is less than 0.01, and according to table 1, have got Performance level of 1, but the walls of 2S-1P-4, 2S-1P-3 for Tabas , Northridge earthquakes and also 2S-1P-3 wall for Bam and Naghan earthquakes is located between safety limit and collapse prevention whereas walls 2S-2P-4, 2S-2P-5, 2S-1P-5 and in some earthquakes the 2S-1P-4 wall have got the total relative displacement between 0.01 and 0.02 which have got limited destruction performance level.

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