Analytical Fragility Curves of Confined Masonry Buildings

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SUMMARY:
In this paper the fragility curves of the two-story regular confined masonry buildings with the rigid ceiling are presented in two levels of limit states corresponding to elastic and ultimate strength versus PGA based on analytical method. In this regard the randomness of parameters indicating the characteristic of the building structure is considered. In order to develop the analytical fragility curves the back bone curves of the analytical models of confined masonry walls for various limit states, introduced in a previous investigation of the authors, are used to specify the damage indices and responses of the structure. In order to obtain damage indices a series of pushover analyses are performed, and to identify the seismic demand a series of nonlinear dynamic analysis are conducted. Finally the fragility curves with assuming a log normal distribution are derived based on capacity and demand of building structures by considering various structural parameters.

Keywords: Two-story regular confined masonry buildings, Analytical models of confined masonry

1. INTRODUCTION

Destructive Earthquakes cause fatality and financial damages in earthquake-prone countries periodically. Therefore in order to risk analysis and retrofitting of structures the seismic assessment of buildings before occurring earthquake could be useful for preventing or decreasing disaster. The fragility curves that represent the probability of exceeding the certain damage versus a seismic intensity parameter, is a suitable tool for the mentioned target. Based on damage data used in the generation of it, the fragility curves can be classified into the four groups of empirical, judgmental, analytical and hybrid resulted from observed post-earthquake surveys, expert opinion, analytical simulation or combination of these respectively. Despite of realistic manner of empirical method, because of the limitation of data the application of empirical fragility curves is very limited. On the other hand if the presented behavioral model is precise, as much as possible, and simple enough to be used by professional engineers, the analytical fragility curves with extensive data can be applied. According to previous studied by authors about confined masonry walls (CMW), the backbone curve of this kind of structure was presented (Ranjbaran et al., 2012).

The proposed model can show the wall behavior before and after cracking. Based on effective factors on confined masonry walls with or without opening, some simple formulas have been proposed to express the relationships between the lateral strength of the confined masonry wall and the wall specifications, including the initial stiffness, the secondary stiffness after cracking, the maximum strength, and ductility, to be used in engineering programs such as SAP, which are widely implemented in engineering firms, by practicing engineers **(**Fig.1.1 & Tab.1.1**).**

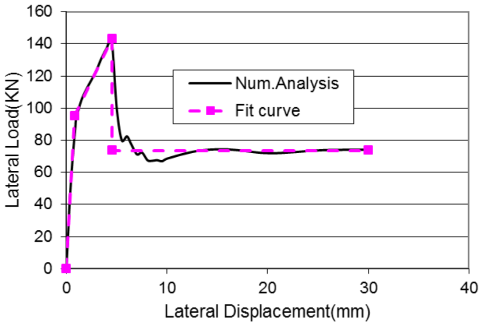

Figure 1.1. A sample of the lateral force-displacement curves of confined masonry walls (Ranjbaran et al. 2012)

By using the proposed analytical formulas it is possible to simulate the CMW buildings in a 3-dimensional configuration by introducing some macro-models in any conventional engineering software for push over and nonlinear dynamic analysis. For this purpose each CMW is substituted by an element of linear configuration, having the geometrical properties of the corresponding wall unit, and a plastic shear hinge at the middle of the substitute element (Ranjbaran et al., 2012,Shiga et al.,1980), whose boundary conditions are defined as a hinge at the bottom, and a moment bearing roller at the top of the element. The plastic behavior of the substitute hinge is given by the proposed formulas. In order to verify the proposed formulas for making macro models of CMW buildings a full scale 2-story building, studied before by Alcocer et al.(1996) under cyclic loading is modeled by the proposed formulas and the results are compared., which shows a good agreement between the two capacity curves and failure mechanism.(Ranjbaran et al., 2012)

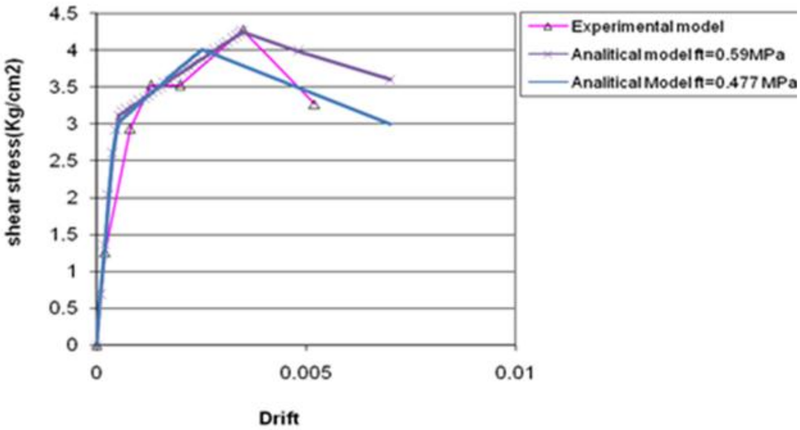

Figure 1.2. Comparison of capacity curves obtained by experimental and numerical model (Ranjbaran et al. 2012

In this paper the fragility curves of the two story levels of regular confined masonry structures with the rigid ceiling versus PGA are presented in two levels of damage limit states corresponding to elastic and ultimate limit strength in the form of an analytical method. In order to develop analytical fragility curves the presented back bone model of confined masonry walls by authors is used for specifying the damage indices and maximum demand.

Table1.1. Analytical formulation for confined masonry walls (Ranjbaran et al., 2012)

	Without Opening	With Opening
K	$\dfrac{1}{[\dfrac{h_w^3}{a\times 3\times E\times I_w}+\dfrac{b\times h_w}{G\times A_w}]}$	$\dfrac{1}{a\times[b^{(\frac{l_o}{l_w}\times\frac{h_o}{h_w})}]\times[\dfrac{h_w^3}{c\times 3\times E\times I_W}+\dfrac{d\times h_w}{G\times A_W}]}$
Q_u	$a\times\left[\mathrm{f_t}\times \mathrm{A_w}\times\dfrac{\mathrm{l_w}}{h_w}\right]^b\times\left[1+\dfrac{\mathrm{f_a}}{f_t}\right]^c$	$a\times\left[\mathrm{f_t}\times \mathrm{A_w}\times\dfrac{\mathrm{l_w}}{h_w}\right]^b\times\left[1+\dfrac{\mathrm{f_a}}{f_t}\right]^c\times\left[\mathrm{d}^{\left(\frac{\mathrm{l_o}}{l_w}+\frac{h_o}{h_w}\right)}\right]$
Q_p	$a\times \mathrm{Q_u}$	$a\times \mathrm{Q_u}$
Q_r	$a\times\left[\dfrac{\mathrm{l_w}}{h_w}\right]^b\times \mathrm{c}^{\mathrm{f_a}}$	$\exp\left[a\times\left(\dfrac{\mathrm{l}_W}{\mathrm{h}_W}\right)^b\times c^{\mathrm{f_a}}\times d^{\left(\frac{\mathrm{l}_o}{\mathrm{l}_W}+\frac{\mathrm{h}_o}{\mathrm{h}_W}\right)}\right]$
D	$[a\times \mathrm{f_t}]+\left[\mathrm{b}\times\left(\dfrac{\mathrm{l_w}}{h_w}\right)\right]+\left[c\times\left(\dfrac{\mathrm{f_a}}{f_t}\right)\right]+d$	$\begin{cases} 0.6\times[5.88-(24.7\times \mathrm{f_a})] & \dfrac{l_p}{h_p}>1 \\ [5.88-(24.7\times \mathrm{f_a})] & 0.75\le\dfrac{l_p}{h_p}\le 1 \\ 1.3\times[5.88-(24.7\times \mathrm{f_a})] & \dfrac{l_p}{h_p}<0.75 \end{cases}$ $\begin{cases} 0.68\times[5.68-(31.32\times \mathrm{f_a})] & \dfrac{l_p}{h_p}>1 \\ [5.68-(31.32\times \mathrm{f_a})] & 0.75\le\dfrac{l_p}{h_p}\le 1 \\ 1.8\times[5.68-(31.32\times \mathrm{f_a})] & \dfrac{l_p}{h_p}<0.75 \end{cases}$

2. FRAGILITY ANALYSIS

Fragility curves in this investigation were derived analytically by proposed analytical models. A three dimension model that represents the ordinary 2 stories confined masonry buildingswas considered for this purpose. The parameters that affect the behavior of confined masonry wall and input motion are considered as random variables.The parameters are: the tensile and compressive strength of unit masonry that influence on mechanical properties of material, and the thickness of walls. Input motions were selected based on the site classification 'B' and the range of horizontal peak ground acceleration (PGA) 0.34-0.36g,that resulted to 3 records from peer strong motion database.The analytical fragilitycurvesare derived in three main steps according to figure2.1(Jeong and Elnashai, 2007):(i)determination of characteristic parameters of structures based on analytical models and pushover analyses of models(capacity curves) (ii)determination the mean of maximum displacements demand based on PGA and nonlinear dynamic analysis of models and(iii)construction of fragility curves with two limit states elastic limit and maximum strength of confined masonry structures.

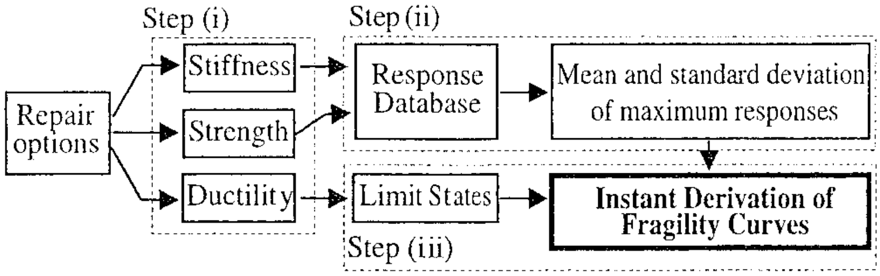

Figure 2.1.Overal procedure of the parameterized fragility curves (Jeong and Elnashai, 2007)

2.1. Reference Structure

Dynamic response history and pushover analysis were performed for 2 stories confined masonry building of clay bricks and rigid diaphragm in ceilings and ties are of the type of concrete based on the recommendations of National Iranian Code of Practice for Seismic Design of Buildings (Standard No. 2800). The plan of building is represented in figure 2.2.

The tensile strength of unit masonry and the thickness of walls were considered as random variables. The tensile strength of unit masonry is very important parameter that affects the behavior of CMW such as the ductility, strength and mechanical properties of CMW's. This value is varied from 0.04 to 0.25 (MPa) (E_m=444 -2778,G_m=178-1111,f_m=0.44-2.78(MPa)) in this investigation that corresponding to cement-sand mortar with ratio 1:12 and 1:6. The thickness of walls is 220 and 350(mm) and horizontal and vertical ties are in the form of reinforced concrete with dimensions of 20×20cm for vertical ties, and 20×20 and 20×35cm for horizontal ties, corresponding to 22 and 35cm walls respectively, the reinforcement inside ties was assumed to be consisted of 4 steel bars of 10mm diameter with the yielding strength of 300Mpa and compression strength of concrete was also assumed to be 15Mpa according to the recommendations of Standard No. 2800. By using proposed analytical formulas and macro model the reference structure was simulated in 3 dimensional models according to previous explanation (Fig.2.3).

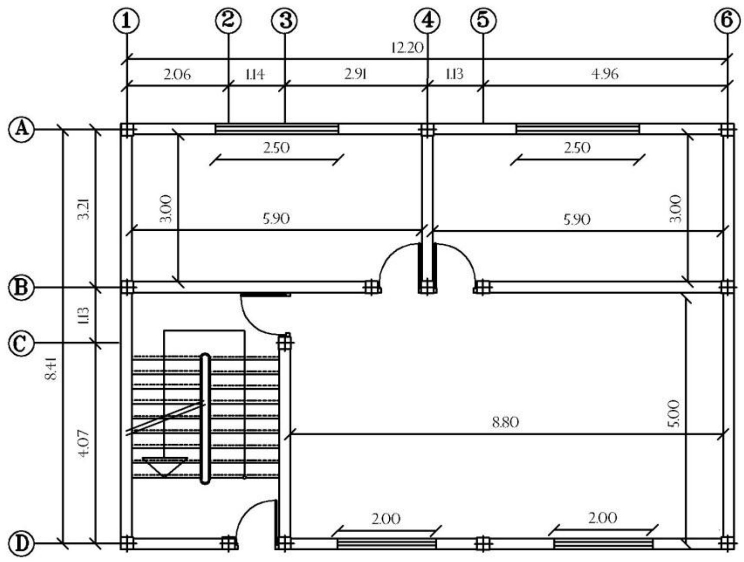

Figure 2.2. Reference structure under analysis

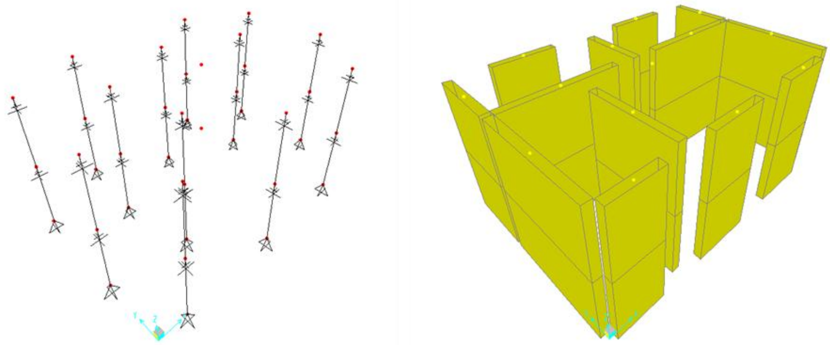

Figure 2.3. Three Dimension of reference structure model

2.2. Demand and Capacity Estimation

Demand estimation in the form of displacement of center of mass in roof is obtained from inelastic dynamic analysis of models for a range of structural parameters.The proposed macro model is used for simulation of reference structure (Fig.2.3) (Ranjbaran et al. 2012). The nonlinear behavior of CMW's is restricted to the shear hinge and its behavior is characterized by the proposed analytical formula with 'Takeda'hysteresis type (Moroni et al 1994, Lumantarna et al 2006). The accelerogram records are obtained from peer Strong motion database with selecting site classification 'B' and the range of peak ground acceleration 0.34 to 0.36g.Three double accelerogram ('NORTHRIDG','NORTHRIDGE, LA - OBREGON PARK' and PARKFIELD) are selected so that represent the ground motion in the firm soil and region with too high relative risk and the significant duration should be at least 10 seconds according to standard No. 2800. Each of double records is applied to models in main directions of structure simultaneously and then the analysis is repeated with changing records in the main directions of structure. The records were scaled to 0.1, 0.25, 0.4 and 0.6g for estimation of displacement demand. The maximum displacement demand is calculated for each PGA among 3 double records in various structural models in the main direction of structure according to standard No. 2800 (Fig.2.4). As a result a database is obtained that represents the maximum displacement corresponding to PGA for various structural parameters in each direction. As indicated in Figure2.5, the means of maximum displacement response from a series of inelastic dynamic analysis can be plotted against PGA (Jeong and Elnashai, 2007). A third order polynomial regression function that represents the PGA as a function of the mean of the maximum displacement demand is used for deriving fragility curves based on PGA.

Capacity estimation of reference structure is obtained by pushover analyses and proposed macro model for various structural parameters in each direction(Fig.2.3&6).As a result a data base is obtained that represents the displacement of limit stats(elastic limit and maximum strength) for various structural parameters in each direction. The mean of these values is considered as capacity value on any of the limit states.For the reference structure the displacement of elastic limit and maximum strength is 4mm and 24.3mm respectively.

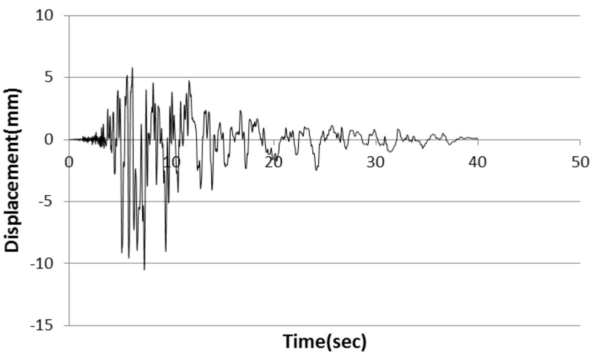

Figure 2.4. A Sample of the response of the reference structure

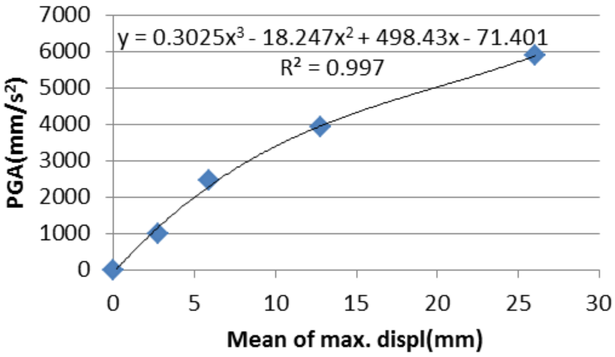

Figure 2.5. Mean of maximum displacements of the reference structure

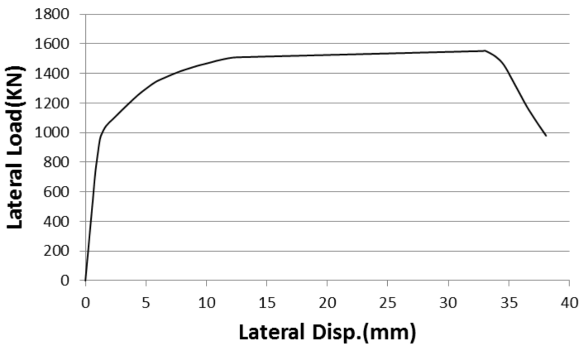

Figure 2.6.A Sample of the capacity curve of the reference structure

2.3. Fragility curves

Using the method that was explained in section 2, the fragility curves of the reference structure are derived for the two limit states corresponding to elastic limit and maximum strength. Based on thedynamic analysis of the models,the probability of maximum demand reaching or exceeding the limit states is calculated and plotted in figure 2.7.

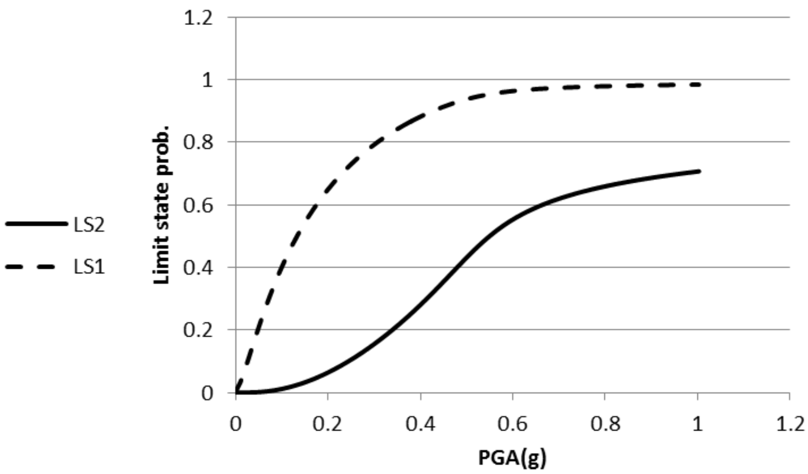

Figure 2.7. Fragility curves (LS1=Elastic limit strength, LS2=Maximum strength)

The probability of reaching or exceeding a limit state at a given PGA can be expressed as follows (Jeong and Elnashai, 2007, Cherng, 2001):

$$P(LS/s)=P[(d_{LS} \leq d_{max})/PGA]=1-\Phi(r) \tag{2.1}$$

Where d_{LS} and d_{max} are limit state capacity and maximum demand, respectively. Assuming that the response follows a log-normal distribution, Φ is the standard normal cumulative distribution and the standard normal r can be expressed as:

$$r = \frac{lnd_{LS}-lnd_{max}}{\sqrt{\beta_{LS}^2+\beta_D^2}} \tag{2.2}$$

β_{LS} and β_D are the lognormal standard deviations of limit state and the displacement demand, respectively.

3. CONCLUSIONS

In this paper the analytical fragility curves versus PGA of a 2-story confined masonry building is presented in two levels corresponding to elastic limit and maximum strength. It is shown that the demand and capacity of the structure with various parameters can be obtained by simulating the confined masonry building by substituting each one of its confined masonry walls with linear element accompanying with a shear hinge at the middle of its length. The behavior of shear hinge is presented by analytical models that can be used for wide range of effective parameters in confined masonry walls with and without opening. The analytical models were verified by experimental models in previous studies. Finally with obtaining capacity and demand of the structure the probability of reaching or exceeding the limit state with assuming log-normal distribution of data becomes possible.
The trend of fragility curves are acceptable and it is shown that with peak ground acceleration equal to 0.13g and 0.55g the probability of reaching the elastic limit and maximum strength is 50 percent respectively.

REFERENCES

Alcocer,S.M.Ruiz,J.Pineda,A.Zepeda,A.(1996). Retrofitting of confined masonry walls with welded wire mesh.Eleventh world conference on earthquake engineering, Elsevier Science, Ltd. paper 1471.

Cherng, R.H.,(2001). Preliminary study on the fragility curves for steel structures in Taipei, Earthquake Engineering and Engineering Seismology,3 ,35-42

Jeong, S.H.,Elnashai, A.,(2007).Probabilistic fragility analysis parameterized by fundamental response quantities. Engineering Structures 29:1238-1251.

Lumantarna, E.,Vaculik, J.,Griffith, M.,Lam, N. and Wilson, J.,(2006).Seismic fragility curves for un-reinforced masonry walls. Earthquake Engineering in Australia, Canberra.

Moroni,M.O.Astroza,M.andTavonatti,S.,(1994).Nonlinear models for shear failure in confined masonry walls.TMS Journal,12(2) :72-78.

Ranjbaran, F., Hosseini, M. and Soltani, S., (2012). Simplified formulation for modeling the nonlinear behavior of confined masonry walls in seismic analysis. International Journal of Architectural Heritage, 6: 259–289.

Shiga,T.Shibata,A.Shibuya,J.andTakahashi,J.(1980).Performance of the building of faculty of engineering, Tohoko University, during the 1978, Miagi-ken-oki earthquake.7th WCEE Istanbul, Vol. 7, p357-364.