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Seismic vulnerability assessment of confined masonry wall buildings

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Abstract. In this paper the vulnerability of the confined masonry buildings is evaluated analytically. The proposed approach includes the nonlinear dynamic analysis of the two-story confined masonry buildings with common plan as a reference structure. In this approach the damage level is calculated based on the probability of exceedance of loss vs a specified ground motion in the form of fragility curves. The fragility curves of confined masonry wall buildings are presented in two levels of limit states corresponding to elastic and maximum strength versus PGA based on analytical method. In this regard the randomness of parameters indicating the characteristics of the building structure as well as ground motion is considered as likely uncertainties. In order to develop the analytical fragility curves the proposed analytical models of confined masonry walls 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 by considering various mechanical and geometric parameters of masonry walls and numerous accelerograms, the fragility curves with assuming a log normal distribution of data are derived based on capacity and demand of building structures in a probabilistic approach

Keywords: confined masonry buildings; analytical models of confined masonry; fragility curves; vulnerability of confined masonry; openSees; DIANA

1. Introduction

Destructive Earthquakes cause fatality and financial damages in earthquake-prone countries periodically. On the other hand, for risk analysis and retrofitting of structures in order to preventing or decreasing disaster, the seismic vulnerability assessment of buildings is required.

Seismic vulnerability assessment and damage scenarios are usually based on macro-seismic method by using macro-seismic intensity hazard map or mechanical methods. In the mechanical methods the seismic vulnerability assessment is acheived by simulating the structural capacity based on mechanical models and represent the hazard scenarios in terms of peak ground

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acceleration or spectral values (Lagomarsino and Giovinazzi 2006). By using the mechanical method, the fragility curves which represent the probability of exceeding of a certain damage state at a seismic intensity measure, are a suitable tool for the mentioned target (Borekci and Kircil 2011). The fragility curves can be used for assessment of performance and economic losses of structures.

Based on damage data which are used in the generation of fragility curves, they can be classified into the four groups of empirical, judgmental, analytical and hybrid, respectively resulted from observed post-earthquake surveys, expert opinion, analytical simulation, and combination of these. Despite of realistic manner of empirical method, the application of empirical fragility curves is very limited due to limitation of data. On the other hand, if the employed behavioral model of the structure is precise, as much as possible, and simple enough to be used by professional engineers, the analytical fragility curves with numerous data can be used for vulnerability assessment of structures.

It has been more than 100 years that confined masonry structures are used in construction. This type of buildings are consisted of masonry walls and confining elements (horizontal and vertical ties), located at four sides of the wall panels. The confining elements are usually made of reinforced concrete (R/C), steel profile, or timbers for increasing the stability, integrity and strength of the masonry wall against seismic in-plane and out-of-plane forces. Many of buildings constructed with confined masonry walls (CMWs) have shown unsatisfactory response in recent earthquakes (Brzev 2007; Moroni *et al*. 2004). On the other hand, the assessment of seismic vulnerability of this kind of structures by experimental or analytical methods is difficult for engineers because of the limited data and simplified models, and usually prescriptive methods are used for this purpose. Ruiz-Garcia and Negrete (2009) presented drift-based fragility curves of CMWs based on empirical method, and some researchers investigated the loss assessment of unconfined masonry buildings based on displacement method, analytically (Ahmad *et al*. 2010, Pagnini *et al*. 2011). In general, the investigation especially in performance assessment of masonry structures is limited due to limited information and data in experimental and numerical studies in this kind of structures.

In this research, based on a previous investigation by authors about numerical and analytical modeling of CMWs with and without openings, it has been attempted to assess the vulnerability of confined masonry buildings in the form of fragility curves based on analytical method. In previous studies of the authors about CMWs the backbone curve of this kind of structures has been presented (Ranjbaran *et al*. 2012). The proposed model can show the wall behavior before and after cracking. Based on the effective factors on the behavior of CMWs with or without opening, some simple formulas have been proposed. These formulas express the relationships between the lateral strength of the CMW and the wall specifications, including the initial stiffness, the secondary stiffness after cracking, the maximum strength, and the wall ductility. By using the proposed analytical formulas it is possible to simulate the CMWs with different mechanical and geometrical parameters as well as various amounts of surcharge in a 3-dimensional configuration by introducing some macro-models instead of CMWs in any conventional engineering software for push over and nonlinear dynamic analysis (Ranjbaran *et al*. 2012, Ahmad *et al*. 2010, Belmouden and Lestuzzi 2009, Penna *et al*. 2013, Magenes and Fontana 1998).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, 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 and macro model two experimental models of CMWs and a full scale 2-story building were modeled by the proposed formulas and the results were compared, which showed good agreement between the capacity curves and also failure mechanism (Ranjbaran *et al*. 2012).

2. Analytical models and verification

The numerical modeling of CMW was performed by DIANA (version 9.3) software. (Ranjbaran *et al*. 2012, DIANA 2005).The continuum finite element method was used, in which masonry wall is simulated in the form of a continuous homogenized with taking into account the orthotropic behavior of material and the interaction between wall and ties was considered.

For verifying the numerical modeling and reducing the model error, the results of modeling were compared with some experimental models. These models include two CMWs (Pourazin and Eshghi 2009, Marinilli and Castilla 2004). Fig. 1 shows the good agreement between numerical and experimental results. In Fig. 1(a) the value of tensile strength of unit masonry (f_t) has been specified in experimental model clearly but in Fig. 1b the experimental model is associated to confined masonry wall with shear strength (v_m) of 0.511 MPa and compression strength (f_m) of unit masonry equal to 6.8 MPa and the tensile strength of unit masonry is not specified . According to experimental model the tensile strength should be between 0.5 to 0.6 MPa (Calderini 2009, Tomazevic 1999).

By modeling many numerical models and considering effective parameters (such as f_t) on the behavior of CMWs, it is possible to develop analytical model with regression analysis on the results of numerical models (Ranjbaran *et al*. 2012). A sample of the fitted curves and analytical formula is presented in Fig. 2 and Table 1 and 2.

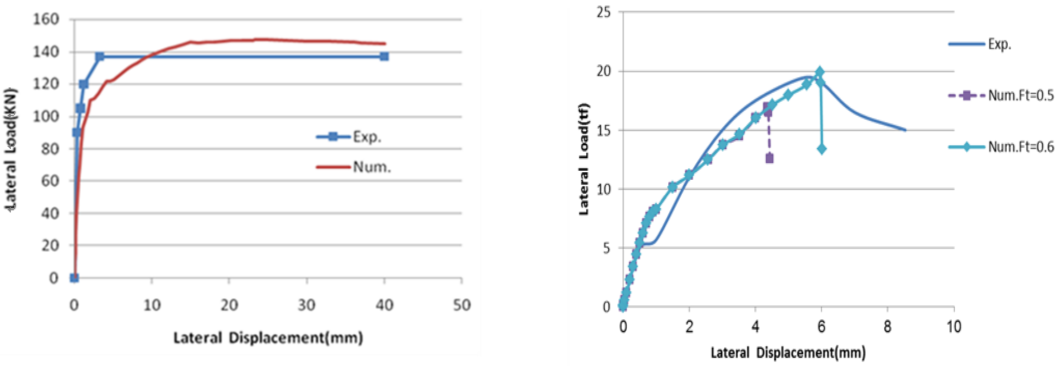
Ideally, two limit states can be considered in the capacity curve:

LS1(Immediate Occupancy):This limit state corresponds to elastic limit strength and the first observable stiffness degradation. This point is the start of transverse cracking at the upper end of vertical tie.

LS2(Collapse Prevention): This limit state corresponds to maximum strength of the wall and the fully formation of diagonal cracking on the surface of the wall.

By using the proposed analytical formulas and macro-model in any conventional software, it is possible to provide the capacity curve of CMW buildings in a 3-dimensional configuration (Ranjbaran *et al*. 2012). In order to verify the proposed formulas for making macro models of CMW buildings a full scale 2-story building under cyclic loading (Alcocer *et al*. 1996) was modeled by the proposed formulas. In the experimental model the shear strength resulting from diagonal compression test and compression strength of unit masonry are 0.59MPa and 5.3MPa respectively. In numerical and analytical model the modules of elasticity and the tensile strength of unit masonry were applied 5300 MPa and between 0.477 to 0.59MPa respectively. The comparison results showed good agreement between the two capacity curves and failure mechanism as shown in Fig. 3 (Ranjbaran *et al*. 2012).

By introducing the backbone curve and appropriate hysteretic behavior of elements it is possible to perform the nonlinear dynamic analysis of structure. In order to simulate the cyclic behavior of masonry walls, degrading stiffness model Takeda-type was employed. This model is suitable for concrete and other brittle material. Ahmad *et al*. (2010) used this model in the



(a) (b)

Fig. 1 Comparison between experimental and numerical models related to the masonry wall with confinement (Ranjbaran *et al*. 2012): (a) Pourazin model (f_t = 0.13MPa), (b) Marinilli model (v_m = 0.511MPa, (0.5 < f_t < 0.6MPa))

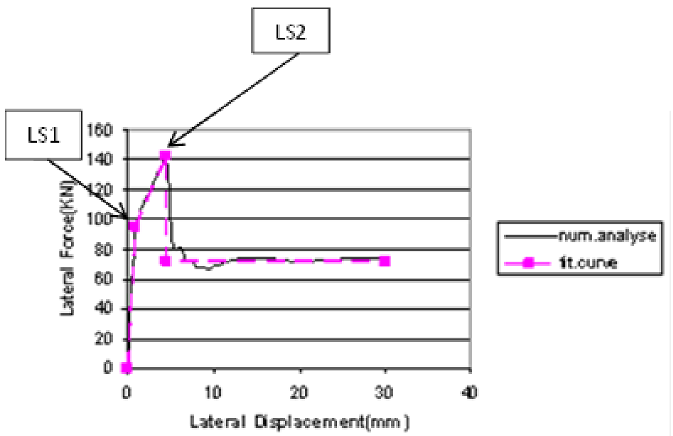


Fig. 2 A sample of the lateral force-displacement curves of CMWs (Ranjbaran *et al*. 2012)

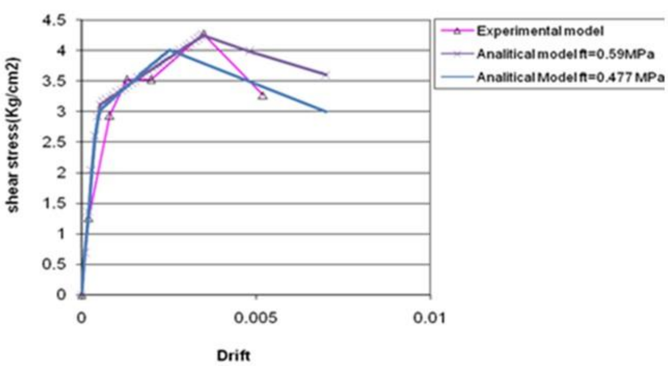


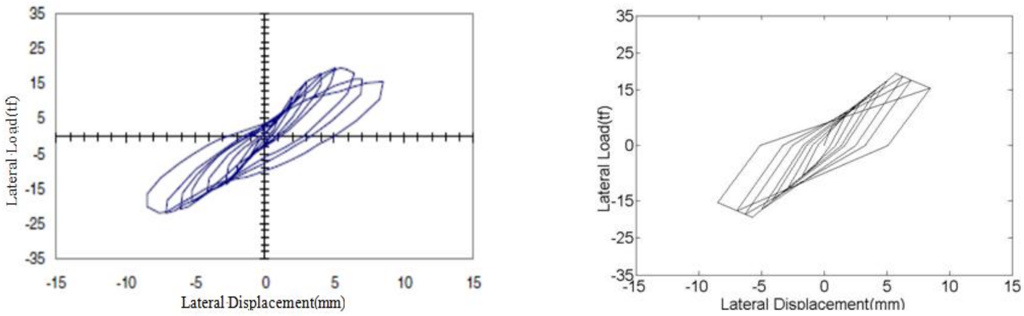
Fig. 3 Comparison of capacity curves obtained by experimental and numerical model (vm = 0.59MPa) (Ranjbaran *et al*. 2012)

Table 1 Analytical formulation for CMWs[N.mm] (Ranjbaran *et al*. 2012)

	Without opening	With opening
K	$\frac{1}{[\frac{h_w^3}{a\times 3\times E\times I_w}+\frac{b\times h_w}{G\times A_w}]}$ (1)	$\frac{1}{a\times[b^{(\frac{l_o}{l_w}\times\frac{h_o}{h_w})}]\times[\frac{h_w^3}{c\times 3\times E\times I_W}+\frac{d\times h_w}{G\times A_W}]}$ (2)
Q_u	$a\times\left[f_t\times A_w\times\frac{l_w}{h_w}\right]^b\times\left[1+\frac{f_a}{f_t}\right]^c$ (3)	$a\times\left[f_t\times A_w\times\frac{l_w}{h_w}\right]^b\times\left[1+\frac{f_a}{f_t}\right]^c\times\left[d^{\left(\frac{l_o}{l_w}+\frac{h_o}{h_w}\right)}\right]$ (4)
Q_p	$a\times Q_u$ (5)	$a\times Q_u$ (5)
Q_r	$a\times\left[\frac{l_w}{h_w}\right]^b\times c^{f_a}$ (6)	$\exp\left[a\times\left(\frac{l_w}{h_w}\right)^b\times c^{f_a}\times d^{\left(\frac{l_o}{l_w}+\frac{h_o}{h_w}\right)}\right]$ (7)
	Without opening	With opening
D	$[a\times f_t]+\left[b\times\left(\frac{l_w}{h_w}\right)\right]+\left[c\times\left(\frac{f_a}{f_t}\right)\right]+d$ (8)	$\begin{cases} 0.6\times[5.88-(24.7\times f_a)] & \frac{l_p}{h_p}>1 \\ [5.88-(24.7\times f_a)] & 0.75\le\frac{l_p}{h_p}\le 1 \\ 1.3\times[5.88-(24.7\times f_a)] & \frac{l_p}{h_p}<0.75 \end{cases}$ $\begin{cases} [0.68\times(5.68-(31.32\times f_a)] & \frac{l_p}{h_p}>1 \\ [5.68-(31.32\times f_a)] & 0.75\le\frac{l_p}{h_p}\le 1 \\ 1.8\times[5.68-(31.32\times f_a)] & \frac{l_p}{h_p}<0.75 \end{cases}$

In the formulas given in Table 1 the following parameters have been used:

K: Initial stiffness
Q_u: Maximum resistance
Q_p: Elastic limit resistance
Q_r: Residual resistance
D: Ductility
E: modulus of elasticity
G: shear modulus
I_w: inertia and cross-sectional area
A_w: area of the horizontal section
h_w: wall height
l_p: Pier Length
h_p: Pier height
l_w: wall length
h_o: opening height
l_o: opening length
f_a: Compression stress on the wall
f_t: Tensile strength of unit masonry



(a) (b)

Fig. 4 Comparison between experimental and numerical models related to the masonry wall with confinement: (a) Experimental model by Marinilli (Marinilli and Castilla 2004), (b) the Numerical model

Table 2 Numerical values of parameters used in Eqs. (1) to (8)

Parameter	Equation No. and the wall thickness (in cm)															
	(1) 22, 35	(2) 22, 35	(3) 22	(3) 35	(4) 22	(4) 35	(5) 22	(5) 35	(5*) 22	(5*) 35	(6) 22	(6) 35	(7) 22	(7) 35	(8) 22	(8) 35
a	1.98	0.95	636	143	505	410	0.76	0.74	0.97	0.95	52338	60567	11.11	11.3	33.88	37.49
b	1.77	237	0.43	0.55	0.45	0.49	-	-	-	-	0.98	0.95	0.064	0.067	-6.39	-6.49
c	-	2.79	0.55	0.83	0.79	0.88	-	-	-	-	18.64	53.58	1.23	1.28	-4.96	-9.00
d	-	1.65	-	-	0.33	0.28	-	-	-	-	-		0.91	0.91	5.62	9.72

*Related to wall with opening

displacement assessment of masonry walls and Vasconcelos *et al*. (2011) showed that this model can capture the cyclic behavior of CMW satisfactorily. For validation of the hysteretic behavior of the proposed macro model, the Marinilli experimental model (Marinilli and Castilla 2004) was simulated in OpenSees v2.3.2 (OpenSees 2009), using the elastic beam-column element for the wall with a nonlinear hinge at the middle of the element in order to modeling the nonlinear behavior of the wall. The hinge was introduced by zero-length element with nonlinear behavior in the lateral translation. The nonlinear behavior was defined using the uni-axial hysteretic material with β = 0.25 (parameter used to determine the degraded unloading stiffness based on ductility) available in the software and the envelope curve of the capacity curve of the CMW. Comparison of results showed that the hysteretic behavior of the proposed macro model has good agreement with experimental model. (Fig. 4).

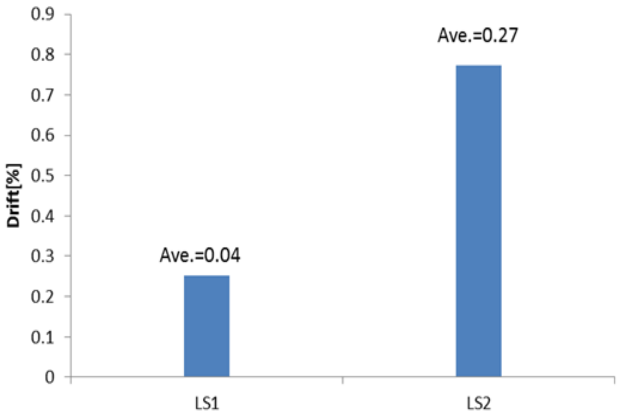


Fig. 5 The drift corresponding to limit states (LS1: Elastc Limit, LS2: Maximum Strength)

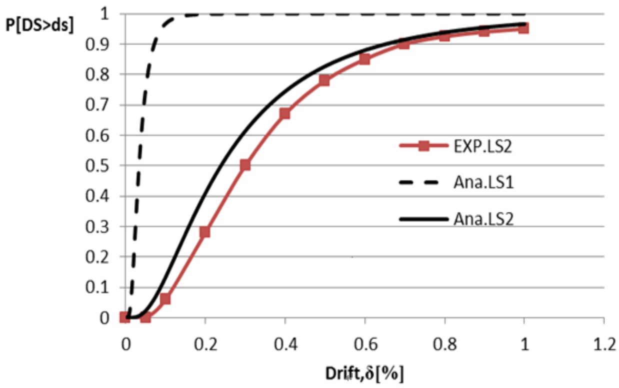


Fig. 6 The drift-based fragility curves corresponding to the maximum strength (LS2) and the elastic limit strength (LS1) for CMWs

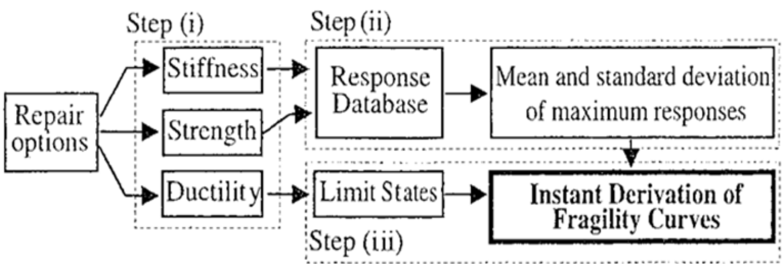


Fig. 7 Overall procedure of the parameterized fragility curves (Jeong and Elnashai 2007)

2. Fragility analysis

The random quantities can be classified in quantitative sources (such as uncertainties in the experimental measures of mechanical and geometrical parameters, seismic demand) and qualitative sources (such as limit states) (Haldar and Mahadevan 2000).In this investigation the fragility curves of confined masonry structures were derived analytically by the proposed analytical models. For this purpose the numerous cases of CMWs with various parameters and a 3-dimensional prototype building were considered. The plan of prototype building represents the ordinary 2-story CMW buildings according to recommendations of National Iranian Code of Practice for Seismic Design of Buildings (Standard No. 2800).The random variables according to above were considered in the analytical models.

2.1 Drift based fragility assessment of CMWs

Drift based fragility curves of more than 600 CMWs without opening were developed for limit state associated with Elastic Limit (LS1) and maximum strength (LS2). For this purpose the tensile strength of unit masonry (f_t) that influences on the mechanical properties of walls, the surcharge (σ_v), the thickness (t) and the aspect ratio of the walls (H/L) were considered as the parameters with uncertainty in CMWs for developing fragility curves. The range of variety of database is presented in Table 3. The tie elements were according to recommendation of Iranian Standard No. 2800. Horizontal and vertical ties were considered in the form of reinforced concrete with dimensions of 20×20 cm for vertical ties, and 20×20 and 20×35 cm for horizontal ties, corresponding to 22 and 35cm walls respectively. The reinforcement inside ties was assumed to be consisted of 4 steel bars of 10 mm diameter with the yielding strength of 300Mpa. Compression strength of concrete was also assumed to be 15Mpa.The bricks are the type of clay brick.

For developing the analytical fragility curves with considering the mentioned uncertainty of parameters the drifts corresponding to elastic and maximum strength of CMWs were calculated by using the proposed analytical models. The average drift corresponding to the elastic limit and the maximum strength were 0.04 and 0.27% respectively (Fig. 5).

According to experimental studies the results of 43 specimens of CMWs tested under lateral cyclic loading during research programs, carried out in Mexico, Chile, Peru, Venezuela and Colombia, the average drifts corresponding to LS1 and LS2 are 0.04% and 0.31% respectively (Ruiz-Garcia and Negrete 2009).These values confirm very well the analytical value of LS1. The experimental models showed a range between 0.23 to 0.31% corresponding to fully formed X shape cracking on the masonry wall surface with V/V_{max}=0.98 and fully formed X shape with concrete crushing at the bottom of tie end columns with V/V_{max}=1 respectively. The analytical value of LS2 is an average value in this range.

In order to develop the fragility curves based on drift the lognormal distribution was employed (see Eq. (1)).

$$P(DS_i > ds_i|\delta) = 1 - \Phi\left(\frac{\ln(\delta) - \mu_{ln\delta_i}}{\beta}\right) \tag{1}$$

where $P(DS_i>ds_i|\delta)$ is the conditional probability of exceeding a certain damage state ds_i in the CMW at a drift value δ, μ and β are the average and standard deviation parameters corresponding to each damage state, and Φ is the standard normal cumulative distribution function (Fig. 6).

Table 3 The range of variables data

f_t (MPa)	σ_v (MPa)	t (mm)	H/L
0.1-0.7	0-0.6	220&350	0.625-1.67

In order to verify the analytical fragility curve, it was compared with experimental results of Ruiz-Garcia and Negrete (2009).The comparison between results of analytical and experimental studies is satisfactory as shown in Fig. 6. The value of β for LS1 and LS2 are 0.59 and 0.78 respectively.

2.2 Fragility assessment of prototype building based on PGA

The fragility curves based on peak ground acceleration (PGA) were developed for limit state associated with the elastic limit (LS1) and the maximum strength (LS2) of CMW buildings. For this purpose a 2-story of regular CMW building with rigid ceilings was considered as the prototype structure. The plan of building and the number of stories is quite popular in the earthquake prone areas. In order to develop analytical fragility curves the back bone model of CMWs, presented by the authors, was used for specifying the damage indices and maximum demand. The parameters that affect the behavior of CMW and input motion were considered as random variables. The analytical fragility curves were derived in three main steps according to Fig. 7 (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 displacement demand based on PGA and nonlinear dynamic analysis of models, and (iii) construction of fragility curves with two limit states of the elastic limit and the maximum strength of CMW buildings.

2.2.1 The prototype structure

Dynamic response history and pushover analyses were performed for the 2-story CMW building made of clay bricks and having rigid diaphragm in ceilings. The ties were assumed to be of concrete type based on the Iranian Standard No. 2800. The plan of building is represented in Fig. 8.

The tensile strength of the masonry unit and the thickness of walls were considered as random variables. The tensile strength of the masonry unit is a very important parameter which affects the features of CMW such as the ductility, strength and mechanical properties (Ranjbaran *et al*. 2012). In this investigation this value is varied from 0.04 to 0.25 (MPa) (E_m = 444 -2778, G_m = 178-1111), corresponding to cement-sand mortar with ratio 1:12 and 1:6 respectively.

The thickness of walls was assumed to be 22 cm and 35 cm, and horizontal and vertical ties were considered 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 35 cm walls respectively. The ties were assumed according to the recommendations of Iranian Standard No. 2800. In both directions of building the density of 22 cm and 35 cm thickness walls was assumed as 5 and 8 percent respectively. By using the proposed analytical formulas and macro model, the reference structure was simulated as a 3-dimensional model (Ranjbaran *et al*. 2012) (Fig. 9).

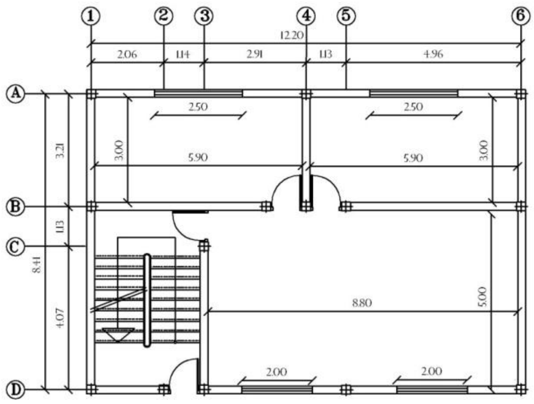


Fig. 8 Reference structure under analysis

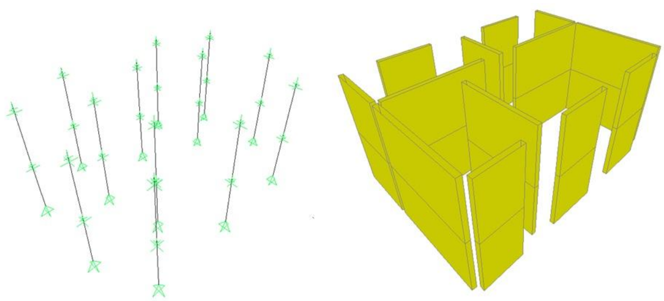


Fig. 9 The 3-dimensional model of the reference structure

2.2.2 Demand and capacity estimation

For the considered range of structural parameters demand estimation in the form of maximum displacement demand of the center of mass in roof was calculated based on the inelastic dynamic analysis of models. The proposed macro model was used for simulation of the reference structure (Fig. 9). The nonlinear behavior of CMWs was restricted to the shear hinge and its behavior was characterized by the proposed analytical formula with 'Takeda' hysteresis type with assuming β = 0.25 based on the best fit between experimental and numerical models (Fig. 4).An ensemble of 7 earthquake ground motions in the form of two-component records (longitudinal and transverse components) were extracted from the PEER Strong Motion Database (Table 4 and Fig. 10). The selected accelerograms have PGA values between 0.3 to 0.4g and their significant duration is at least 10 seconds, and they have been recorded on firm soil site (site classification 'B'[USGS]) with generally main period less than 0.3sec (Fig. 11). It is believed that these conditions result to threatening conditions for typical masonry constructions because of both frequency content and the area with high relative risk.

Each of two-component records was applied to models in main directions of structure simultaneously and then the analysis was repeated with changing records in the main directions of structure (for three-component records the results did not change significantly). The records were scaled to 0.1, 0.25, 0.4, 0.5 and 0.6g for estimation of displacement demand. In the various

Table 4 Selected earthquake records

Earthquake	Distance(Km)	Station
SAN FERNANDO	24.2	24278
VICTORIA, MEXICO	34.8	6604
WHITTIER NARROWS	22.5	14403
LOMA PRIETA	13	58065
NORTHRIDGE	29	90021
NORTHRIDGE	9.2	24087
CHI-CHI	33.01	TCU047

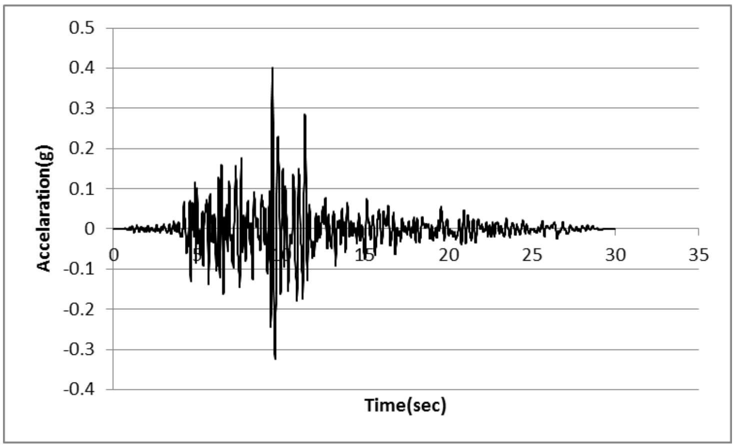


Fig. 10 'NORTHRIDGE (90021)' accelerogram

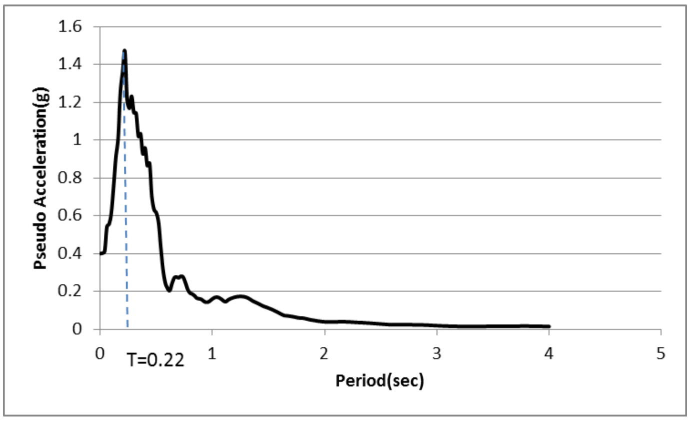


Fig. 11 The Acceleration spectrum of NORTHRIDGE (90021) record

structural models and in the main direction of structure the maximum displacement demand is calculated for each PGA for 7 two-component records (Fig. 12) which results to ensemble data base of 560 data (Fig. 13).

According to experimental studies for CMWs of building with rigid slabs which exceed the maximum resistance, no damage is observed to the walls orthogonal to seismic excitation of confined masonry wall (Tomazevich and Klemenk 1997), this is translated to about 1.5-3g in terms of accelerations (Vaculik *et al*. 2007). Therefore, by satisfying the recommendation of Iranian Standard No. 2800, the assumption of elastic behavior of the walls orthogonal to seismic excitation is acceptable. Also the verification of analytical model confirms this subject (Ranjbaran *et al*. 2012). In the time history analyses damping ratio was considered 5% according to the previous researches (Flores and Alcocer 1996, Tomazevich and Klemenk 1997).

As indicated in Fig. 14, the mean values of maximum displacement response from a series of inelastic dynamic analysis can be plotted against PGA (Jeong and Elnashai 2007). A forth order polynomial regression function that represents the mean of the maximum displacement demand as a function of the PGA is used for deriving fragility curves based on PGA (Fig. 14).

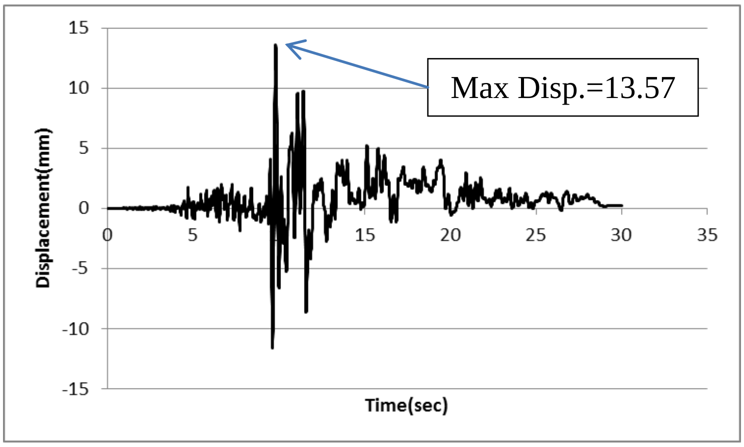


Fig. 12 A Sample of the response of the reference structure

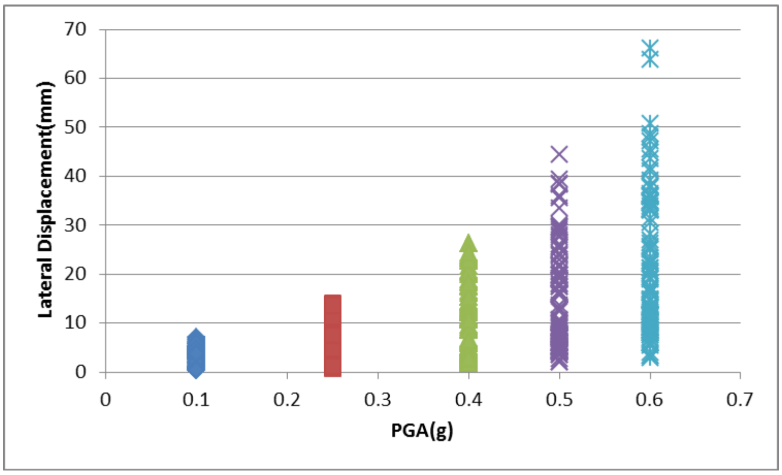


Fig. 13 The maximum displacement demands for 7 two-component records

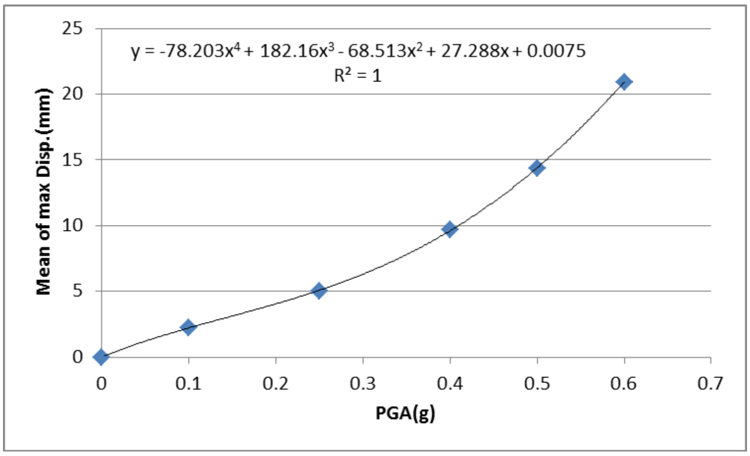


Fig. 14 Mean values of maximum displacements of the reference structure vs the PGA

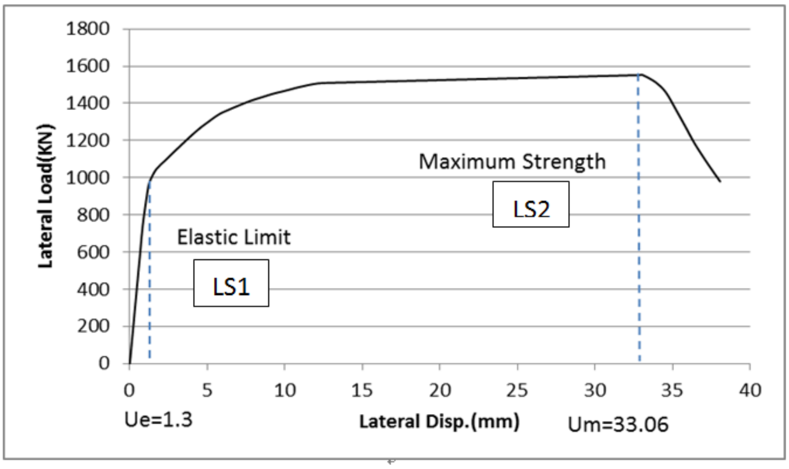


Fig. 15 A Sample of the capacity curve of the reference structure

In the next step of development of fragility curves by using the proposed macro model for various structural parameters the capacity estimation of reference structure was obtained by pushover analyses in each direction (Fig. 15) (Ranjbaran *et al*. 2012). As a result, a database was obtained which represents displacement of limit states (elastic limit and maximum strength) for various structural parameters in each direction. The mean of these values was considered as capacity value for each of the limit states. For the reference structure the displacement of elastic limit and maximum strength was 4 mm and 24.3 mm respectively.

2.3 Fragility assessment

By using the method that was explained in Section 2.2, the fragility curves of the reference structure were derived for the two limit states corresponding to elastic limit (LS1) and maximum strength (LS2) with respect to PGA. Based on the nonlinear dynamic and pushover analysis of the

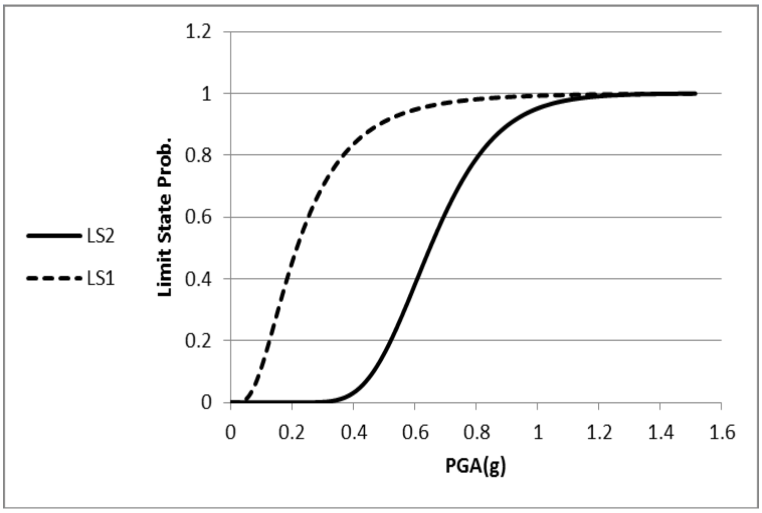


Fig. 16 Fragility curves (LS1=Elastic limit strength, LS2=Maximum strength)

models, the probability that the maximum demand reaches or exceeds the limit states is calculated and plotted in Fig. 16.

The probability of reaching or exceeding a limit state at a given PGA can be expressed as Eqs. (2)-(3) (Jeong and Elnashai 2007, Cherng 2001):

$$P(LS/s) = P[(d_{LS} \leq d_{max}) \,/\, PGA] = 1 - \varphi(r) \tag{2}$$

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

$$r = \frac{lnd_{LS} - lnd_{max}}{\sqrt{\beta_{LS}^2 + \beta_D^2}} \tag{3}$$

β_{LS} and β_D are the lognormal standard deviations of limit state and the displacement demand, respectively (Pagnini *et al*. 2011). The discrete probabilities were transformed to continuous form by using lognormal probability paper. The fragility curves are obtained by using lognormal probability paper.(Boreckci and Kircil 2011, Karim and Yamazaki 2001).In this investigation the parameters of distribution λ and ζ (mean and standard deviation) for limit states of LS1 and LS2 are equal to 7.66,0.63 and 8.76,0.26 respectively. The experimental studies have shown that the maximum resistance of confined masonry buildings is relatively high. The experimental model of Tomazevich and Klemenk (1997) showed that the three-story confined masonry buildings will be able to withstand, with limited damage to the walls (according to the maximum strength of building), a strong earthquake PGA value of 0.8g, and will be prevented from collapse when subjected to PGA values exceeding 1.3g. It should be mentioned that the experimental models are in accordance with the requirement of EC8. In this study the reference structure is in accordance with the requirement of the Iranian Standard No. 2800 and the fragility curve shows that with PGA equal to 0.65g the probability of reaching the maximum strength is 50 percent.

3. Conclusions

In this paper the analytical fragility curves of the various CMWs and a 2-story building with likely uncertainties versus drift and PGA were presented in two levels corresponding to elastic limit and maximum strength. For CMWs because of the existence of empirical fragility curves corresponding to maximum strength, the compatibility between empirical and analytical fragility curves was achieved. Then according to previous studies the demand and capacity of the CMW building can be obtained in terms of both uncertainties of the ground motion and characteristics of structure, this is achieved by simulating the confined masonry building by substituting each of its CMWs with linear element and 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 CMWs with and without opening. The analytical models were verified by experimental models in present and previous studies. Finally, by obtaining capacity and demand of the structure it makes possible to derive the probability of reaching or exceeding the limit state with assuming the log-normal distribution of data.

The trend of fragility curves are acceptable and the curves show that with peak ground acceleration equal to 0.22g and 0.65g, the probability of reaching the elastic limit and maximum strength is 50 percent respectively. Results of the study also show that if the CMW buildings satisfy the requirement of the National Iranian Code of Practice for Seismic Design of Buildings (Standard No. 2800) the performance of this kind of structure even against severe earthquakes can be satisfactory.

References

Ahamad, N., Crowley, H., Pinho, R. and Ali, Q. (2010), “Displacement-based earthquake loss, Assessment of masonry buildings in Mansehra City”, Pakistan, *J. Earthq. Eng.*, **14**(S1), 1-37.

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

Belmouden, Y. and P. Lestuzzi (2009), “An equivalent frame model for seismic analysis of masonry and reinforced concrete buildings”, *Construct. Build. Mater.*, **23**(1), 40-53

Boreckci, M. and Kircil, M. (2011), “Fragility analysis of R/C frame buildings based on different types of hysteretic model”, *Struct. Eng. Mech.*, **39**(6), 795-812.

Brzev, S. (2007), *Earthquake-Resistant Confined Masonry Construction*, Kanpur, India: National information center of earthquake engineering (NICEE).

Calderini, C., Cattari, S. and Lagomarsino, S. (2010), “The use of diagonal compression test to identify the shear mechanical parameters of masonry”, *Construct. Build. Mater.*, **24**, 677-685.

Cherng, R.H. (2001), “Preliminary study on the fragility curves for steel structures in Taipei”, *Earthq. Eng. Eng. Seismol.*, **3**, 35-42

DIANA (2005), *DIANA Finite Element Analysis, user’s Manual-Element Library*, Delft: The Netherland :TNO Building and Construction Research.

Flores, L.E. and Alcocer, S.M. (1996), “Calculated response of confined masonry structures”, *In Proceedings of the Eleventh World Conference on Earthquake Engineering, Mexico*, Elsevier Science, Ltd. paper 1830.

Haldar, A. and Mahadevan, S. (2000), “Probability, reliability and statistical methods in engineering design”, Wiley, New York.

Jeong, S.H. and Elnashai, A. (2007), “Probabilistic fragility analysis parameterized by fundamental response quantities”, *Eng. Struct.*, **29**, 1238-1251.
Karim, K.R. and Yamazaki, F. (2001), “Effect of earthquake ground motions on fragility curves of highway bridge piers based on numerical simulation”, *Earthq. Eng. Struct. Dyn.*, **30**, 1839-1856
Lagomarsino, S. and Giovinazzi, S. (2006), “Macroseismic and mechanical models for the vulnerability and damage assessment of current buildings”, *Bull. Earthq. Eng.*, **4**, 415-443.
Marinilli, A. and Castilla. E. (2004), “Experimental evaluation of confined masonry walls with several confining columnsz”, *In Proceedings of the Thirteenth World Conference on Earthquake Engineering*, Vancouver, Canada, August 1-6, 2004, paper 2129.
Magenes, G. and Fontana, A. (1998), “Simplified non-linear seismic analysis of masonry buildings”, *Proceeding Br. Masonry Society*, No. 8: 190-195
Moroni, M.O., Astroza, M. and Tavonatti, S. (1994), “Nonlinear models for shear failure in confined masonry walls”, *TMS J.*, **12**(2), 72-78.
Moroni, M.O., Astroza, M. and Acevedo, C. (2004), “Performance and seismic vulnerability of masonry housing types used in Chile”, *J. Perform. Constr. Facil.*, **18**(3),173-179.
OpenSees, (2006), “Open sees command language manual, Pacific, Earthquake Engineering Research center”, University of California, Berkeley.
Pagnini, L., Vicente, R., Lagomarsino, S. and Varum, H. (2011), “A mechanical model for the seismic vulnerability assessment of old masonry buildings”, *Earthq. Struct.*, **2**(1), 25-42
Penna, A., Lagomarsino, S. and Galasco, A. (2013), “A nonlinear macroelement model for the seismic analysis of masonry buildings”, *Earthq. Eng. Struct. Dyn.*, DOI: 10.1002/eqe.2335
Pourazin, K. and Eshghi, S. (2009), “In plane behavior of a confined masonry wall”, *Masonry Soc. J.*, **27**(1), 21-34
Ranjbaran, F., Hosseini, M. and Soltani, S. (2012), “Simplified formulation for modeling the nonlinear behavior of confined masonry walls in seismic analysis”, *Int. J. Architect. Herit.*, **6**, 259-289.
Ruiz-Garcia, J. and Negrete, M. (2009), “Drift-based fragility assessment of confined masonry walls in seismic zones”, *J. Eng. Struct.*, **18**,170-181.
Shiga, T., Shibata, A., Shibuya, J. and Takahashi, J. (1980), “Performance of the building of faculty of engineering”, Tohoko University, Miagi-ken-oki earthquake, 7th WCEE Istanbul, **7**, 357-364.
Takeda, T., SOZEN, M.A. and Nielsen, N.N. (1970), “Reinforced concrete response to simulated earthquake”, *J. Struct.Eng. Div.*, ASCE, **96**(12), 2257-2573.
Tomazevich, M. and Klemenk, I. (1997), “Verification of seismic resistance of confined masonry buildings”, *Earthq. Eng. Struct. Dyn.*, **26**(10), 1073-1088
Tomazevic, M. (1999), *Earthquake Resistance design of masonry buildings*, Imperial College press.
Vaculik, J., Lumantarna, E., Griffith, M., Lam, N. and Wilson, J. (2007), “Dynamic response behavior of unreinforced masonry walls subject to out of plane loading”, *Australian Earthquake Engineering Society conference*, Wollongong, NSW.
Vasconcelos, G., Polettil, E., Salavessa, E., Jesus, A., Lourenco, P. and Pilaon, P. (2011), “In plane shear behavior of traditional timber walls”, SHATIS, 11, *International Conference on Structural Health Assessment of Timber Structures*, Lisbon Portugal.

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