AUT Journal of Civil Engineering

AUT J. Civil Eng., 5(2) (2021) 225-244
DOI: 10.22060/ajce.2021.18224.5666

Seismic Performance Assessment of Confined Masonry Buildings Based on Displacement

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ABSTRACT: Confined masonry buildings exist or are constructed in some countries due to low cost, simplicity of construction, easy access to materials, and no need for a professional workman. Experience from past earthquakes shows that poor quality of construction and nominal designs are some reasons for the seismic vulnerability of this kind of structure. One of the nominal values in the codes for the design of masonry buildings is the density of masonry walls or the ratio of the area of walls to the area of the plan regardless of the structural features. In this study using the existing constitutive behavior of confined masonry walls and nonlinear static analysis in the form of Acceleration-Displacement Response Spectra (ADRS), the various plans with different densities of walls as prototypes models are assessed for design spectrum presented in standard No. 2800 with a return period of 475 years. For this purpose, the maximum drift of walls at the performance-point of models is compared with various limit states. The results show that based on the value of wall density the seismic behavior of models varies from linear to nonlinear response and the determination of density of walls in the confined masonry buildings for each limit state depends on the characteristics of the wall and several stories. This method could be recommended for the performance-based design of confined masonry buildings.

Review History:

Received: Apr. 06, 2020
Revised: Jan. 06, 2021
Accepted: Jun. 16, 2021
Available Online: Jun. 20, 2021

Keywords:

Confined masonry
Displacement based
Performance point
Numerical model
Wall density

1- Introduction

The confined masonry structures (CMS) are constructed for more than 100 years. Easy access to materials, low cost of construction, no need for a professional workman, high strength of confined masonry walls, and satisfactory performance in past earthquakes are some of the reasons for the construction of this kind of structure. This kind of structure consists of masonry walls that are confined by concrete or steel or timber tie elements. The concrete tie elements are similar to concrete columns or beams in reinforced concrete structures but with smaller cross-sections and several armatures in the cross-section. Usually, the role of tie elements in confined masonry walls (CMW) is to increase the ductility and integrity of masonry walls.

Experience from past earthquakes shows that one of the reasons for the seismic vulnerability of this kind of structure is due to weak strength resulting from inadequate shear strength because of inadequate number or area of masonry walls in the plan or inadequate shear strength of mortar (v_m). This vulnerability appears in the form of shear or sliding failure, to prevent it, the density of walls is proposed in the codes.

A comprehensive study was done in Chile after the 1985 Llolleo earthquake. Experience from the earthquake shows that most damages were inflected to medium-rise buildings (3- to 5-story high), low rise buildings sustained very limited damage (2-story high) and none of the single-story buildings were damaged. Also, it seems that the extent of damage in masonry buildings was related to wall density. Buildings with a wall density of less than 0.5% sustained severe damage, while the buildings with a wall density of 1.15% sustained only light damage [1].

Eurocode8 prescribes the required wall density as follows [2]:

a) At least 2% for a site with a design ground acceleration up to 0.2g.

b) At least 4% for a site with a design ground acceleration up to 0.3g.

c) At least 5% for a site with a design ground acceleration up to 0.4g.

Franch and his colleagues showed that to prevent damage in confined masonry structures the wall density must be greater than 1.15% when the seismic intensity is VIII in the Mercalli modified scale. A relationship was obtained between the value of wall density as proposed index and the observed damage of CMS in the march 1985 central Chile subduction earthquake (MS=7.8) in the form of a case study [3].

Some researchers have studied the seismic behavior of confined masonry structures using the concept of performance-based assessment [4-6]. It is believed that the CMSs have ductile behavior to earthquake excitation and so the damage index based on displacement can be defined for

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them.

Alcocer and his colleagues defined three limit states for confined masonry walls built with hand-made clay brick. Serviceability at the drift of wall 0.15% with the criterion of onset of masonry inclined cracking, repair ability at the drift of wall 0.25% with the criterion of inclined cracking fully formed over masonry wall, hairline cracking into tie-columns and onset of masonry crushing and safety at the drift of wall 0.4% with the criterion of shear strength of the wall, wall cracking penetrates tie ends, yielding of tie-column reinforcement due to shearing and tie-column crushing [4].

Based on experimental evidence derived from tested CMWs under in-plane lateral cyclic loading, a relationship between the increases of lateral drift, the evolution of damage, and structure degradation was established. Seven limit states are presented and four of them are explained here. The drift of 0.04% and the ratio of shear to the maximum shear strength of 0.5 accompany with the formation of the flexural hairline horizontal cracking and hairline vertical cracking near the vertical tie end and the level of damage in this stage is light, the drift of 0.13% and the ratio of shear equal to 0.85 with the first diagonal cracking due to diagonal tension in the masonry wall surface and the damage level of moderate, the drift of 0.23% and the shear ratio of 0.98 which fully formed “X-shape” cracking on the masonry wall surface and the level of damage is heavy in this stage and the drift of 0.42% with the shear ratio of 0.99 and the severe damage level, concentrated diagonal cracking and concrete spalling occurs at the end of tie-columns [7].

Bilgin and Huta studied the earthquake performance assessment of 5-story confined masonry buildings with a tensile strength of masonry 0.27 MPa and thickness of outer walls of two first stories are 380mm and inner walls are 250 mm remaining stories are 250 mm thick. They recommended performance levels corresponding to damage limit states on the capacity spectrum for masonry buildings. No-damage (LS1), Slight (LS2), moderate(LS3) and extensive damage(LS4) The performance criteria for each performance level are: (LS1) Damage is not observed in the walls and the behavior is elastic (LS2) the structure can be utilized after the earthquake without any need for significant strengthening and repair with drift limit of 0.1%, (LS3) the building cannot be used after the earthquake without significant repair and strengthening is feasible with drift limit of 0.3% and (LS4) repairing the building is neither possible nor economically reasonable and the structure will have to be demolished after the earthquake, beyond this limit state the structure goes to collapse, the drift limit for this performance level is 0.5%. They concluded that the drift limit in the case study does not exceed 0.3% [8].

Ranjbaran and Hosseini studied the fragility curves of confined masonry walls numerically. Drift-based fragility curves of more than 600 CMWs without opening were developed for limit states associated with Elastic Limit (LS1) and maximum strength (LS2). LS1 corresponds to the drift of 0.04% and formation of the first observable stiffness degradation and the onset of hairline transverse cracking at the upper end of vertical ties. LS2 corresponds to the drift of the wall 0.27%, this limit state corresponds to the maximum strength of the wall and the full formation of diagonal cracking in the wall [6].

Eshghi and Pourazin studied the nonlinear response of one-story confined masonry buildings with 220 mm thickness of walls based on performance point. They concluded that the one-story confined masonry buildings in very high seismicity regions (PGA=0.35 g) and with masonry tensile strength of 0.15 MPa perform at immediate occupancy level such that the drift angle of walls is less than 0.15% [9].

The proposed values for the density of the walls are empirical without considering the tensile or shear strength of masonry, surcharge of the walls, regularity or irregularity of the plan, the number and distance between the walls in each direction, or the limit states of damage in the walls. Generally, it is recommended at least 2% of wall density is required in each direction of the building to ensure good seismic performance [1], but the features of the building are not considered. The seismic assessment of the confined masonry buildings based on wall density by experimental method consumes time and is costly; on the other hand, due to the lack of numerical methods, it is less done numerically. Some researchers have studied the masonry buildings numerically which resulted in complex yield surfaces that almost preclude the use of modern numerical algorithms and an accurate representation of inelastic behavior [10]. Lourenco and his colleagues presented a constitutive behavior model for unreinforced walls. In this method, masonry wall is modeled as a continuous homogenous and anisotropic medium (continuous finite element) with Rankine-type’s criterion is used as the yielding criteria in tension and Hill-type’s criterion in compression [10]. This proposed model is acceptable in terms of precision; however, its usage is very time-consuming and costly. Some numerical and analytical studies have been conducted due to simplification and saving time and cost in the form of a proposed macro model instead of the proposed continuous finite element which is usable in conventional software and called the equivalent-frame modeling method. [7, 11-15].

In this paper, the adequacy of prescription of Iranian standard #2800 [16] based on the minimum required wall density is studied numerically. The proposed simplified analytical approach and prototype models as common CMS plans are used in their various limit states.

2- Regulations and Assumptions

According to the National Iranian Code of Practice for Seismic Design of Buildings (Standard No. 2800) [16], the thickness of walls must not be less than 22 cm for confined masonry buildings in the first and second stories and 35 cm for an underground story, also the value of masonry walls density with unit masonry of clay brick are as follow (Table 1):

For evaluation of the adequacy of the minimum required wall density according to standard No. 2800, some parameters which can affect the seismic behavior of CMS are

Table 1. The density of masonry walls according to standard No. 2800 [16].

No. of stories	Seismicity of Zone					
	Very High and High			Medium and low		
	underground	1st story	2nd story	underground	1st story	2nd story
1	6%	4%	-	5%	3%	-
2	8%	6%	4%	6%	5%	3%

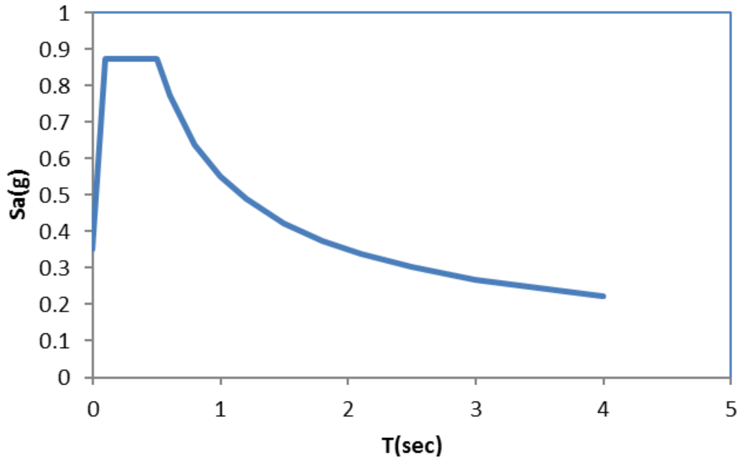


Fig. 1. Standard design spectrum (Standard No.2800) [16].

taken into account. These parameters are period of building, building aspect ratio, Number of confined masonry walls in direction of the earthquake, the thickness of walls, tensile and compression strength of masonry, regularity, and irregularity of plan and number of stories. Some assumptions in this study are: rigid diaphragm in ceiling, the height of stories is 3m, wall failure is in the form of shear failure which is the dominant form of failure in the CMW [17], it should be mentioned that in the prototype models the ratio of height to length of the walls (H/L) is less than one and flexural deformation is not considered in this study [5], clay brick as unit masonry, sand mix cement as mortar material, peak ground acceleration (PGA) equals to 0.35g as a very high level of seismic hazard, concrete tie elements as confining elements with properties according to the prescription of standard#2800, the thickness of walls 22 and 35 cm, design spectrum is according to standard design spectrum of standard#2800 (elastic design spectrum) with a return period of 475 years and probability of exceedance of 10% in 50 years and the type of soil is type II (firm soil) with shear wave velocity (Vs) between 375-750 m/s which is the threatening condition for typical CMS (Fig. 1). In the case of buildings with very slender walls and large spans, weak diaphragms, or weak connections between walls, it is expected that the out-of-plane deformation of walls is significant [18]. On the other hand, the out-of-plane response of walls is considered negligible concerning the global building response dominated by their in-plane response [8]. In this study because of the rigid diaphragm and tie elements as confining elements and geometrical features of walls which are under standard No.2800 the out-of-plane behavior of walls is disregarded [19].

The type of analysis is the nonlinear static analysis (pushover analysis). The capacity curve of CMS is obtained and then the capacity spectrum method (Method “A” in ATC-40) is used to find a performance point or demand in the ADRS (acceleration–displacement response spectrum) format [20]. Based on proposed limit states for confined masonry walls, the seismic performance of prototype models can be assessed. The results of this investigation with considering the assumptions of the study can predict the seismic performance of one and two-story confined masonry buildings based on wall density and the material quality of masonry walls for the region with very high hazard level (PGA=0.35 g) and the firm soil (type II) as the base ground.

3- Numerical Modeling

In this study for assessing the seismic performance of CMSs based on displacement, the equivalent-frame method is employed according to proposed analytical models for CMW (Tables 2 and 3) [6]. This analytical model is developed by the continuum finite element method using DIANA software (version 9.3) [21]. In this software, the

Table 2. The analytical formulation for CMWs (N.mm) [6].

	Without Opening	With Opening
K	$\dfrac{1}{[\dfrac{h_w^3}{a\times 3\times E\times I_w}+\dfrac{b\times h_w}{G\times A_w}]}$ **(1)**	$\dfrac{1}{a\times[b^{(\frac{l_o}{l_w}\times\frac{h_o}{h_w})}]\times[\dfrac{h_w^3}{c\times 3\times E\times I_W}+\dfrac{d\times h_w}{G\times A_W}]}$ **(2)**
Q_u	$a\times\left[f_t\times A_w\times\dfrac{l_w}{h_w}\right]^b\times\left[1+\dfrac{f_a}{f_t}\right]^c$ **(3)**	$a\times\left[f_t\times A_w\times\dfrac{l_w}{h_w}\right]^b\times\left[1+\dfrac{f_a}{f_t}\right]^c\times\left[d^{\left(\frac{l_o}{l_w}+\frac{h_o}{h_w}\right)}\right]$ **(4)**
Q_p	$a\times Q_u$ **(5)**	$a\times Q_u$ **(5)**
Q_r	$a\times\left[\dfrac{l_w}{h_w}\right]^b\times c^{f_a}$ **(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)**
D	$[a\times f_t]+[b\times\frac{l_w}{h_w}]+\left[c\times\frac{f_a}{f_t}\right]+d\ \geq 1$ **(8)**	For 22 cm $1\leq\begin{cases}0.6\times[5.88-(24.7\times f_a)] & \frac{l_p}{h_p}>1\\ [5.88-(24.7\times f_a)] & 0.75\leq\frac{l_p}{h_p}\leq 1\\ 1.3\times[5.88-(24.7\times f_a)] & \frac{l_p}{h_p}<0.75\end{cases}$ For 35 cm $1\leq\begin{cases}0.68\times[5.68-(31.32\times f_a)] & \frac{l_p}{h_p}>1\\ [5.68-(31.32\times f_a)] & 0.75\leq\frac{l_p}{h_p}\leq 1\\ 1.8\times[5.68-(31.32\times f_a)] & \frac{l_p}{h_p}<0.75\end{cases}$ **(9)**

In the formulas given in Table 2, 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: Elasticity modulus of masonry
G: Shear modulus of masonry
f_t: Tensile strength of masonry
l_p: Pier Length
h_p: Pier height
l_w: wall length
h_o: opening height
l_o: opening length
h_w: wall height
Aw: area of the horizontal section
Iw: Moment inertia of cross-section area
f_a: Compression stress on the wall

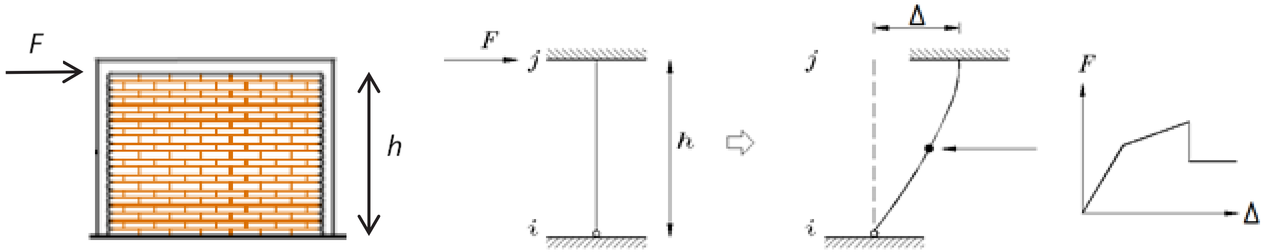


Fig. 2. Proposed Macro and analytical model for confined masonry wall [6].

Table 3. Numerical values of parameters used in Eqs. (1) to (8) [6].

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

masonry walls are simulated in the form of a continuous homogenized environment and Rankin-type's criterion and Hill-type's criterion are used for expressing the inelastic behavior in tension and compression, respectively [10]. In the equivalent-frame method, each confined masonry wall in the direction of an earthquake is modeled into a beam-column element as a macro-model with geometrical and mechanical properties similar to those of masonry walls, and the nonlinear behavior of a confined masonry wall (post cracking) is modeled by a shear hinge at the mid-span of the macro-model with characteristic behavior according to proposed analytical formulas (Tables 2 and 3, and Fig. 2). The boundary conditions of the macro-model are the hinge and fixed- roller at the bottom and above of the element, respectively.

The modeling of prototype models in the form of 3-D using proposed analytical formulas was carried out by OpenSees software [22]. It should be mentioned that the tie elements are taken into account in the proposed analytical models in the way that the masonry wall and tie elements are considered together in the proposed macro model (Fig. 2). The proposed analytical models include the thickness of masonry walls 22 and 35cm and the horizontal and vertical ties in the form of reinforced concrete type with dimensions of 20×20 cm^2 for 22 cm of wall thickness and 20×35 cm^2 for 35 cm of wall thickness, also the compressive strength of concrete is equal to 15 MPa (MPa), and the longitudinal reinforcement is 4Φ10 with the yielding strength of 300 MPa according to Iranian Standard #2800.

Table 4. Mechanical properties of the material [19].

Masonry Compressive strength (*fm*, MPa)	Masonry Tensile strength (*ft*, MPa)	Masonry Compressive Fracture Energy (*Gfc*, N, mm/mm^2)	Masonry Tensile Fracture Energy (*Gft*, N, mm/mm^2)	Masonry Plastic Strain of the peak compressive strength (κ_p)	Concrete compressive strength (fc, MPa)	Yielding stress of armatures (*fy*, MPa)
16.38	0.13	29.33	0.016	0.000645	28	340

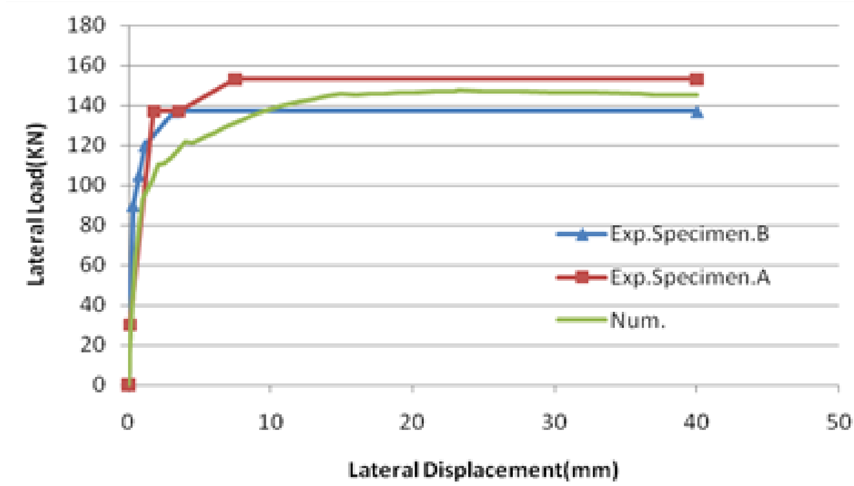
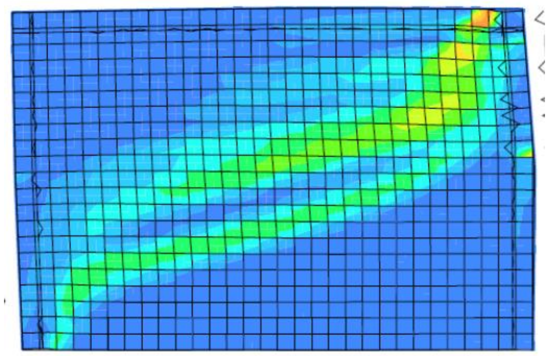


Fig. 3. Comparison between the capacity curve of experimental and numerical models [6, 19].

(a)



(b)

Fig. 4. Comparison between fracture mechanism of the experimental and numerical models; (a) Experimental model, (b) Numerical model [6, 19].

3- 1- Validation

For verifying the analytical modeling developed based on the behaviors explained in the previous section, the results of numerical modeling were compared with some experimental models [6,14,15]. These models include confined masonry walls for verifying the modeling of the compound system of masonry walls and ties. Lateral loading was monolithically applied to the numerical models, at first the vertical loading and after that, the lateral loading was applied separately.

3- 1- 1- Pourazin and Eshghi model [19]

Experimental models of Pourazin and Eshghi [19] include two confined masonry walls (A and B) with length and height of 2.42 m and 1.735 m, respectively, the thickness of wall 21 cm and section of horizontal and vertical tie elements are 21×21 cm^2 accompany with 4Φ10 as longitudinal reinforcement. The mechanical properties of material resulting from the test are presented in Table 4. There is no vertical load on the models. Figs. 3 and 4 compare the capacity curve and the fracture

Table 5. Mechanical properties of the material [23].

Masonry Compressive strength (*fm*, MPa)	Masonry shear strength (*vm*, MPa)	Compressive strength of mortar (MPa)	Concrete compressive strength (*fc*, MPa)	Yielding stress of armatures (*fy*, MPa)
6.8	0.511	7.01	23.8	447.3

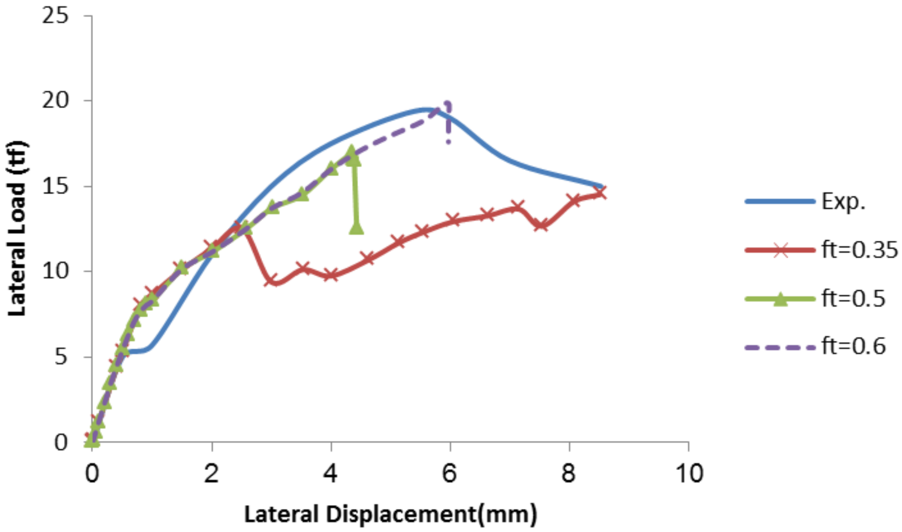


Fig. 5. Comparison between the capacity curve of experimental and numerical models [6, 23].

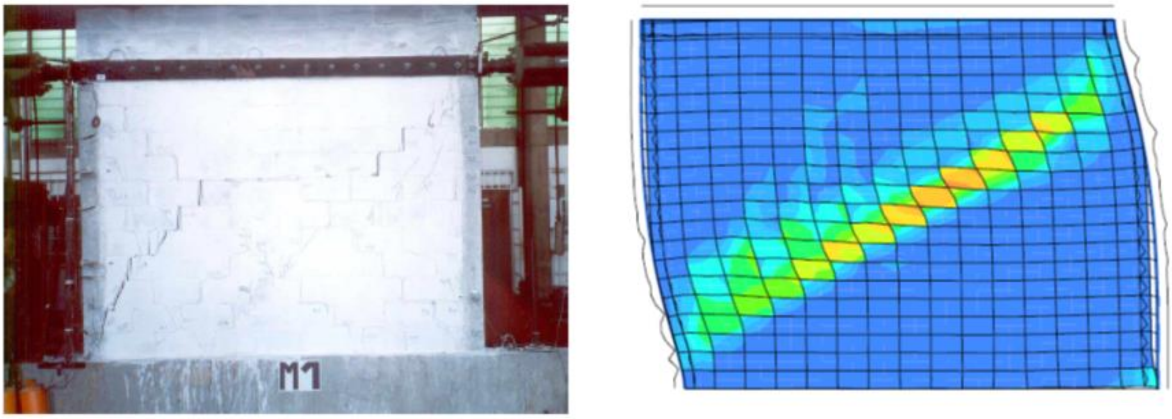
mechanism between experimental and numerical models and also the extent of cracking in two models. As it is shown there is a good agreement between them.

3- 1- 2- Marinilli and Castilla model [23]

Another case is related to the experimental study of Marinelli and Castilla [23]. The experimental model is the confined masonry wall with length and height of 3 m and 2.3 m, respectively, the thickness of the wall is 15 cm and the section of horizontal and vertical tie elements are 15×20 and 15×15 cm^2, respectively which were reinforced lengthwise with 4 No. 4 bars. The mechanical properties of material resulting from tests are presented in Table 5. A vertical load of 13.8 (t_f) is applied to Model A.

The tensile strength of masonry (has not been reported in the experimental study. Flores and Alcocer studied the hysteresis rules of confined masonry walls based on experimental studies with average masonry compressive and diagonal compression strength between 5.4, 3.5, and 0.52, 0.25 MPa, respectively (They showed that with changing of these parameters the maximum lateral strength of the wall changes considerably in comparison with the elastic limit strength. So it seems the tensile strength of masonry is one of the most important parameters that affect the maximum lateral strength of the wall. On the other hand, the determination of the mechanical properties of masonry by the adequate testing method is an important part of the verification of the load-bearing capacity and stability of masonry structures. Generally, precise determination of mechanical properties of masonry such as tensile strength of masonry, modulus of elasticity or shear modulus of masonry, cohesion or friction coefficient between a unit of masonry are difficult and the reliability of them by some methods such as diagonal compression test depends on the degree of anisotropy of masonry which depends on some uncertainties such as the arrangement of unit masonry, the aspect ratio of unit masonry or the thickness of mortar [24], so these parameters are considered based on the proposed range in the previous studies. It seems that the tensile strength of masonry and modules of elasticity are dependent on compression strength of masonry (f_m), some researchers proposed a specific range for these parameters, $200f_m \leq E_m \leq 2000f_m$ and $0.03f_m \leq f_t \leq 0.09f_m$ [25] or $0.025f_m \leq f_t \leq 0.1f_m$ [24].

Therefore, in this experimental study, it is assumed that $0.2 \leq f_t \leq 0.61$ MPa. On the other hand, the shear strength of masonry resulting from the diagonal compression test was reported 0.511 MPa, by assuming the assimilation of diagonal tensile strength and tensile strength of masonry (=) [24], it seems the value of between 0.5 and 0.6 MPa is acceptable for the numerical model. The comparison of the capacity curve and failure mechanism is shown in Figs. 5 and 6. Fig. 5 shows the effect of tensile strength of masonry



(a) (b)

Fig. 6. Comparison between fracture mechanism of the experimental and numerical models; (a) Experimental model, (b) Numerical model [6, 23].

Table 6. Range of variables included in the experimental and analytical study [5, 6].

	f_t(MPa)	v_m(MPa)	σ_v(MPa)	H/L
Experimental	-	0.19-0.98	0-1.37	0.625-1.25
Analytical	0.1-0.7	-	0-0.6	0.625-1.67

on the maximum lateral strength of the wall, it is seen that with increased tensile strength of masonry, there is a good agreement between the capacity curves. Also, a diagonal cracking failure (shear failure) is observed in two studies.

3- 1- 3- Ruiz-Garsia and Negrete studies [5]

To check the accuracy of the values of the limit states based on proposed analytical models corresponding to drift of elastic limit strength (LS1) that the first observable stiffness degradation and the onset of hairline transverse cracking at the upper end of vertical ties occurs in CMW and drift of maximum strength (LS2) that corresponds to fully formation of diagonal cracking in the walls, the results of 43 specimens of CMWs by experimental studies [5] are compared with 600 specimens by analytical models [6]. The experimental models are under lateral cyclic loading during research programs, carried out in Mexico, Chile, Peru, Venezuela, and Colombia. For this purpose the tensile strength of masonry (f_t) or diagonal compression strength of masonry (v_m) that influence the mechanical properties of the walls, the surcharge (), and the aspect ratio of the walls (H/L) were considered as the parameters with uncertainty in CMWs for developing fragility curves. The unit masonry of the two studies is clay brick and the range of variables is presented in Table 6.

The average drifts corresponding to LS1 and LS2 based on experimental and proposed analytical models are 0.04%, 0.31%, and 0.04%, 0.27%, respectively.

Experimental studies confirm the analytical value of LS1 very well. The experimental models showed a range of drift between 0.23 to 0.31% corresponding to fully formed X shape cracking on the masonry wall surface and fully formed X shape with concrete crushing at the bottom of tie end columns, respectively. On the other hand, the analytical value of LS2 is in an average of this range.

To better compare, the analytical fragility curve based on the drift of walls and corresponding to LS2 was developed and compared with the empirical fragility curve [5,6]. The comparison between the results of analytical and experimental studies is satisfactory as shown in Fig. 7.

3- 1- 4- Alcocer and his colleagues model [26]

The last case is related to the push-over analysis of an experimental full-scale model of a 2-story building under cyclic loading [26] that the capacity curve and failure mechanism are compared with the proposed analytical model (Fig. 8a). By using the proposed analytical formulas and macro-model in any conventional software (OpenSees software [22]), it is possible to provide the capacity curve of CMW buildings in a 3-dimensional configuration [6]. Lateral load pattern was applied proportional with the story masses multiplied by their height above the foundation level. In the experimental model, the shear strength resulting

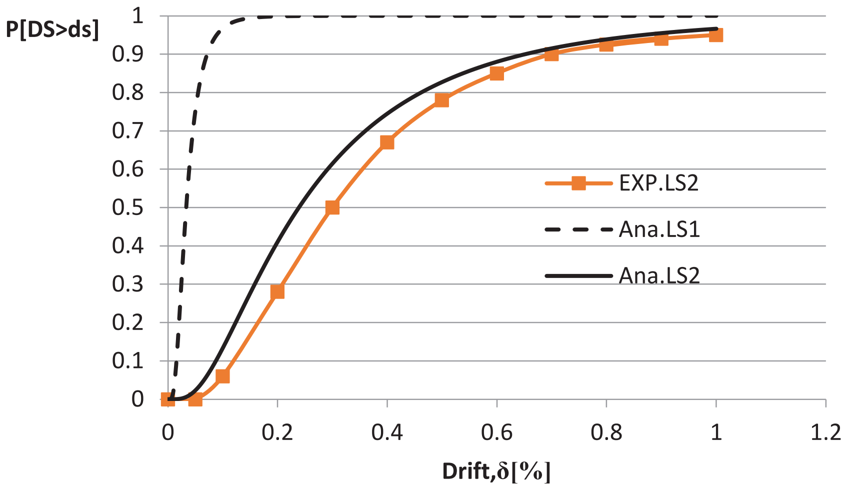


Fig. 7. The drift-based fragility curves correspond to the maximum strength (LS2) and the elastic limit strength (LS1) for CMWs [5, 6].

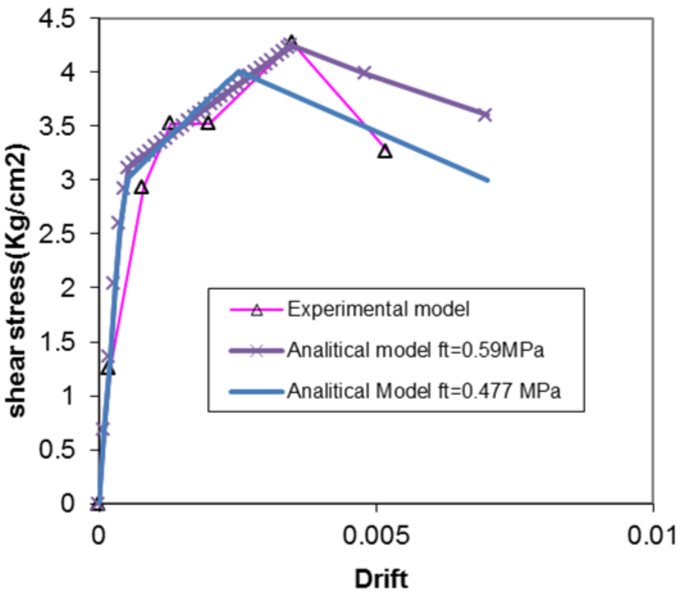
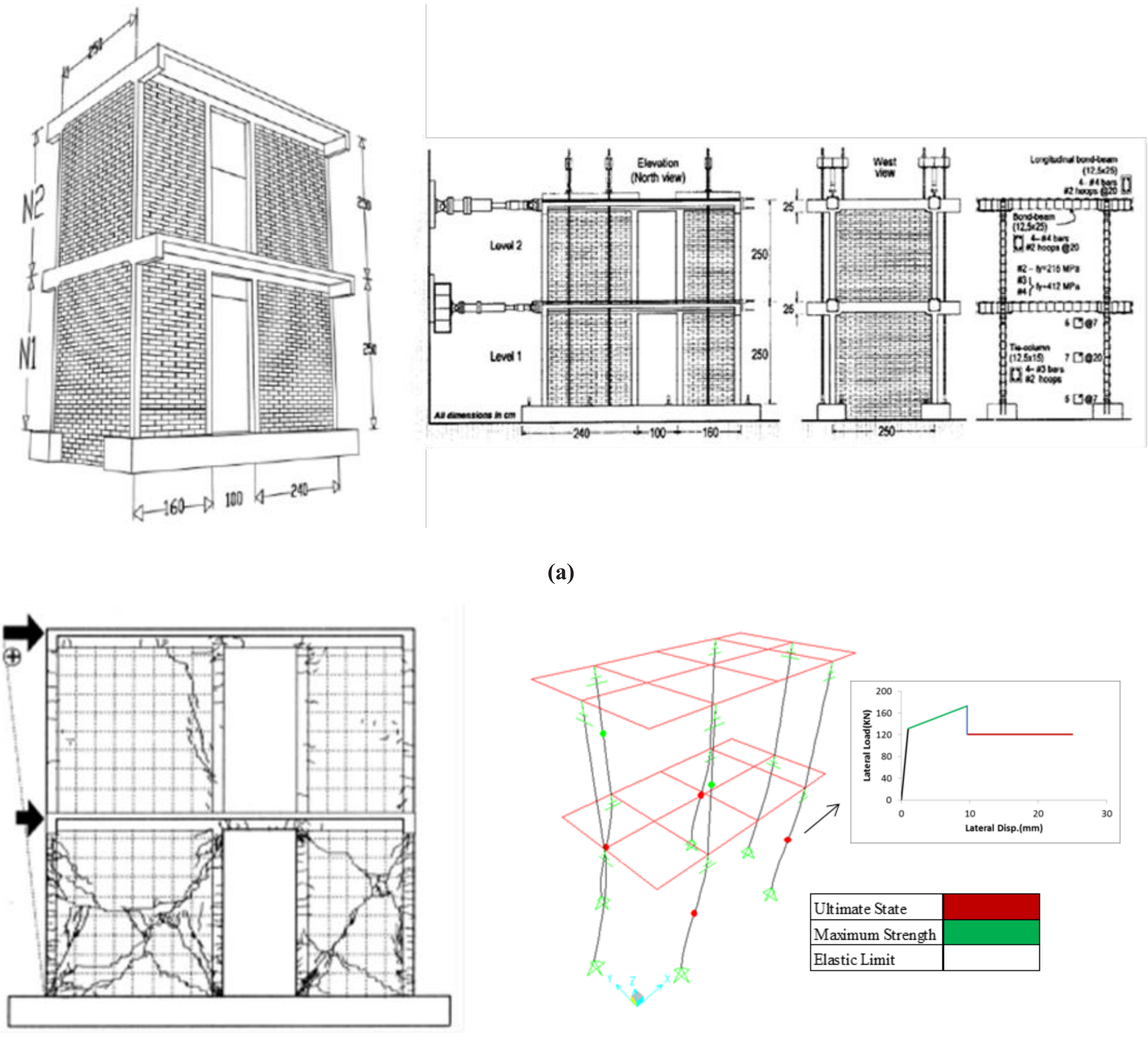


Fig. 8. The comparison of the capacity curve between experimental and numerical models [6, 26].

from the diagonal compression test and compression strength of masonry are 0.59 and 5.3 MPa, respectively. The value of modulus of elasticity and tensile strength of masonry is not available in an experimental study, so in this study based on descriptions in sec 3-1-2, the modules of elasticity and tensile strength of masonry were considered 5300 MPa (E_m=1000f_m (MPa) [8]) and 0.477, 0.59 MPa, respectively. The last value was assigned to the tensile strength of masonry considering the shear strength of masonry resulting from the diagonal compression test [8]. The results showed good agreement between the two capacity curves and damage mechanism as shown in Figs. 8 and 9 (b and c). As it is seen the damage is concentrated in the walls of the first story.

3- 2- Prototype Models

In this study, 3 types of plans as common plans for masonry buildings are considered. The plans with areas 40, 82, and 102 m^2 which named as models 1, 2, and 3, respectively (Figs. 10 to 12). Models 1, 3, and 2, 3 are considered as 1and 2-story, respectively. The thickness of walls and tensile strength of masonry are the varied parameters to assess their effect on the performance level. The thickness of walls and tensile strength of masonry are 22 and 35 cm, and 0.1, 0.15, and 0.25 MPa, respectively. The range of modulus of elasticity and shear modulus are (1111-2778) MPa and (444-1111) MPa, respectively. The height of stories is 3 m and the type of ceilings is the rigid diaphragm. The dead and live load on each story is 500 and 200 kg/m^2, respectively.



(a)

(b)

(c)

Fig. 9. The comparison of damage mechanism between experimental and numerical study; (a) 3-D experimental model [26], (b) Damage status in the experimental model [26], and (c) Damage status and distribution of plastic hinge by numerical model [6]

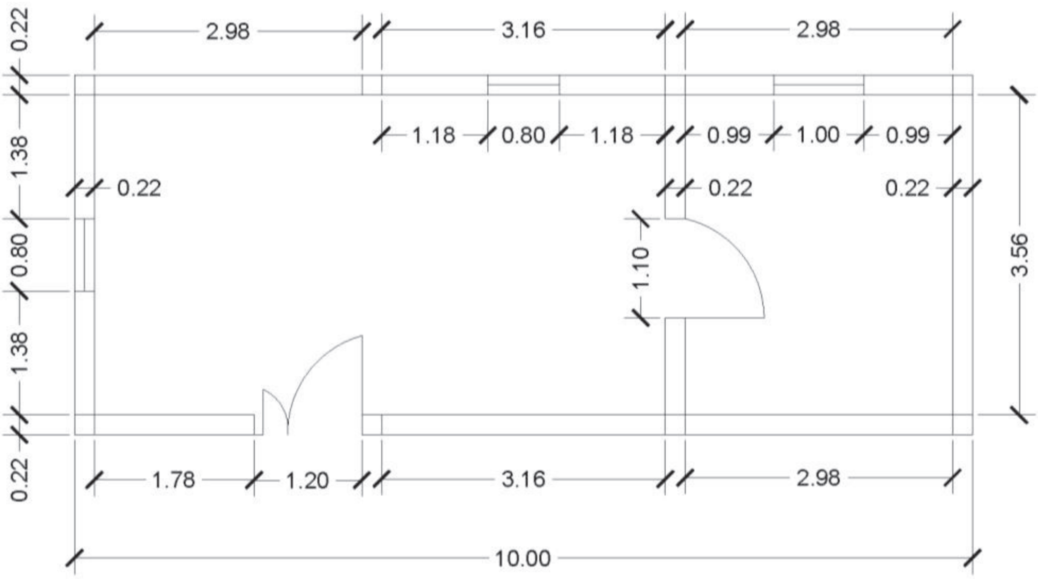


Fig. 10. Plan with 40 m2 (model 1) [9].

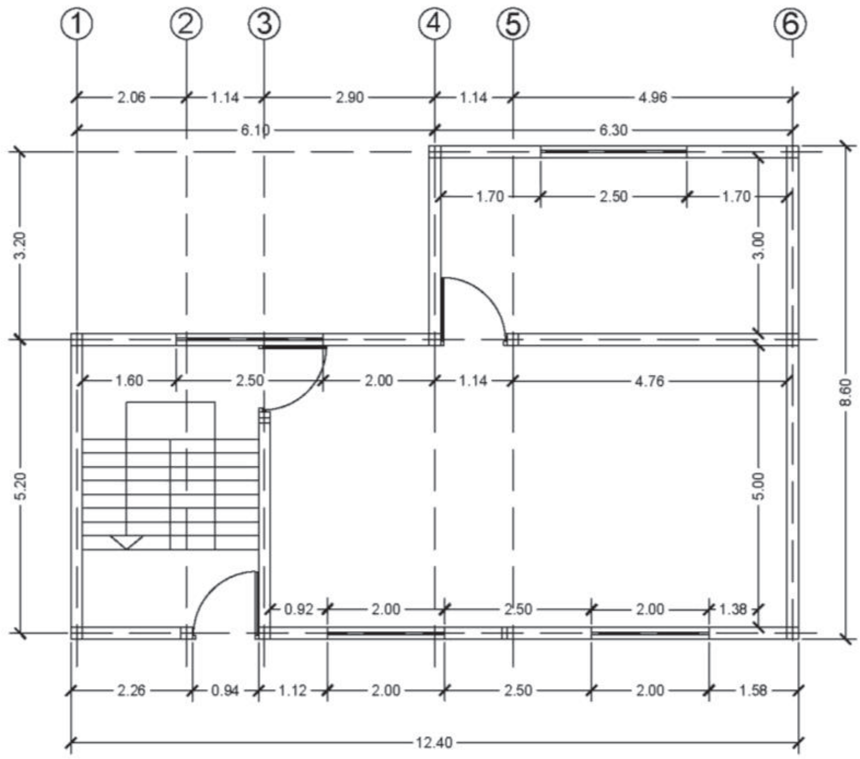


Fig. 11. Plan with 82 m2 (model 2).

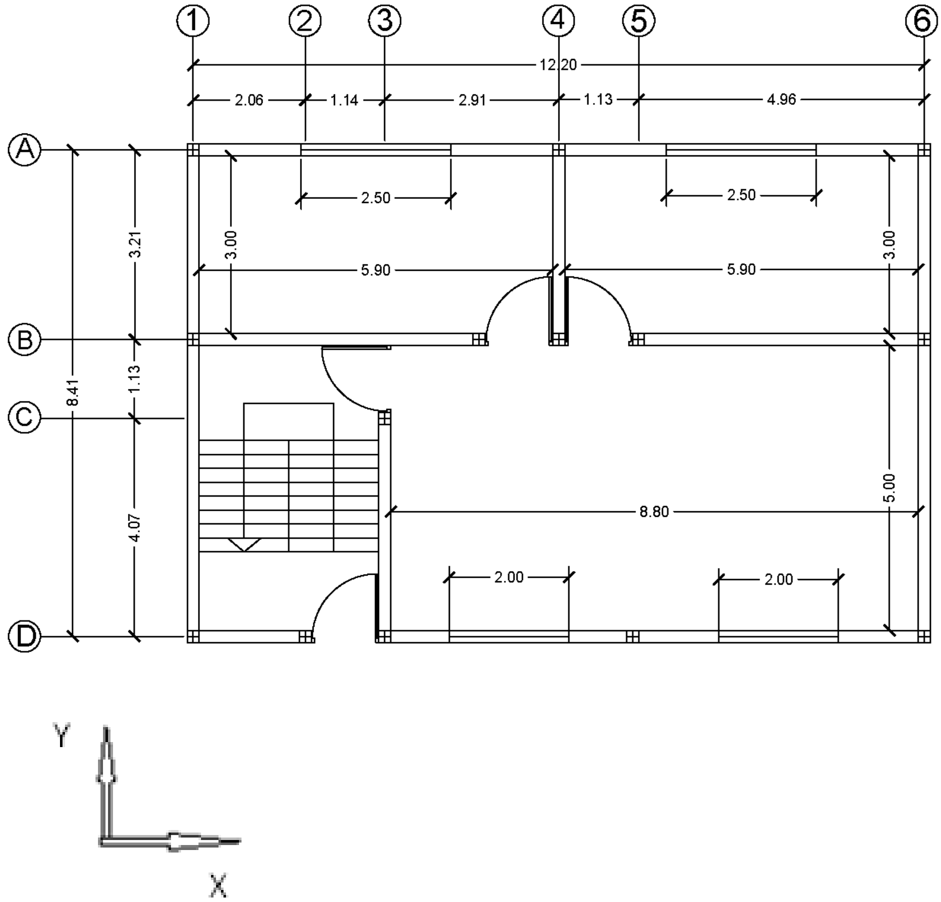


Fig. 12. Plan with 102 m2 (model 3).

3- 3- Analytical Procedure

In this study, the procedure for assessing the seismic behavior and performance point of the prototype models is nonlinear static analysis in the ADRS format. According to section 3, the capacity curve of prototype models is obtained by modeling them in OpenSees software (2.4.0). In this modeling each CMW is simulated by an equivalent linear element "Elastic Beam-Column" and related post-cracking behavior is modeled by a shear hinge at the mid-span of the element. The shear hinge in software is simulated by "element zero Length" and its backbone curve by "uniaxial material hysteretic". The backbone curve of the shear hinge is assigned by the analytical model developed by the author previously [6] and presented in Tables 2 and 3. The "Rigid Diaphragm" command is used to model the ceilings. For a sample, the idealized model of the model- 3 is presented in Fig. 13. For analysis, a displacement control analysis (pushover analysis) was performed by a target displacement of 3cm (0.01h) based on the height of the first story (3m) and a lateral load pattern was applied proportional with the story masses multiplied by their height above the foundation level [27]. It seems the triangular load pattern provides conservative results, at least when buildings are regular in elevation [28].

The capacity spectrum method is used to find a performance point. The elastic design spectrum is the standard design spectrum of Iranian standard No.2800 with a return period of 475 years in the region of very high hazard and with 5% damping, the site condition is the firm soil (Type II) (Fig. 1). Method "A" in ATC-40 is used to find the performance point in two main directions of the prototype models [20]. To get the performance point, the capacity curve and the demand curve must be drawn in one figure, and the intersection of these two curves shows the performance point of the structure.

In this method after transforming the capacity curve and demand curve into a response spectral ordinate, both 5% damped elastic design spectrum and capacity spectrum are plotted on the same chart. Then the first choice of spectral acceleration and displacement (Ai, Di) is selected based on equal displacement approximation or engineering judgment, and then the spectral reduction factors are calculated(SR_A, SR_V), after that the intersection point of reduced demand spectrum and capacity spectrum is determined (Aj, Dj) again, if this point is within an acceptable range related to the previous

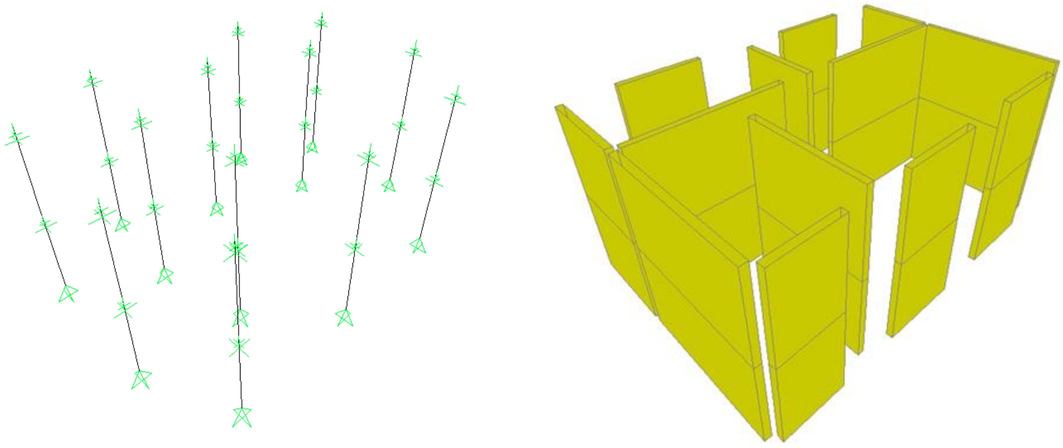


Fig. 13. The idealized model of "model 3".

Table 7. Iteration to find the performance point.

Iteration No.	D_i (cm)	A_i	βeff	D_j (cm)	A_j	SR_A	SR_V	Difference (%)
1	0.29	0.526	12%	0.46	0.59	0.72	0.78	36
2	0.31	0.536	13%	0.4	0.57	0.69	0.76	22
3	0.33	0.544	14.9%	0.34	0.54	0.64	0.73	3

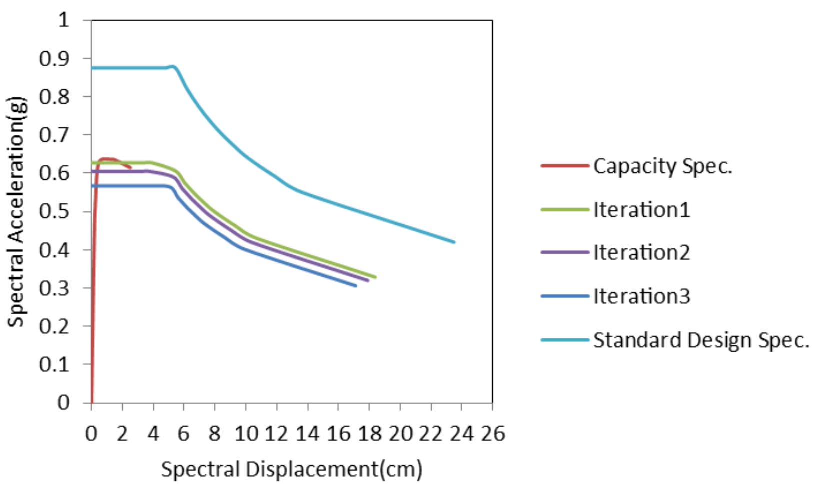


Fig. 14. Performance point according to the capacity spectrum procedure (ADRS).

pointthe performance point will be determined, otherwise the iteration is continued until it reaches an acceptable range. One of the parameters that should be determined is the estimation of damping. The reduction factor of 5% damped design spectrum (SR_A, SR_V) is specified based on this parameter. The damping (βeff) is the combination of viscous damping (5%) and hysteretic damping (βo) from Eq. (8) in ATC-40 [20], in this formula κ-factor has been introduced to modify the hysteretic damping which depends on structural behavior. "Type B" represents a moderate hysteretic behavior and κ of 2/3are assigned for the confined masonry structure [29]. For a sample, Fig. 14 and Table 7 show this procedure for the prototype model with area 102 m^2 and 2-story with the tensile strength of masonry (f_t) of 0.25 MPa and wall thickness of 35 cm at X direction.

4- Results and Discussion

According to previous studies on the confined masonry buildings, the damage in CMS is concentrated in the first story of the building and the failure is similar to soft story

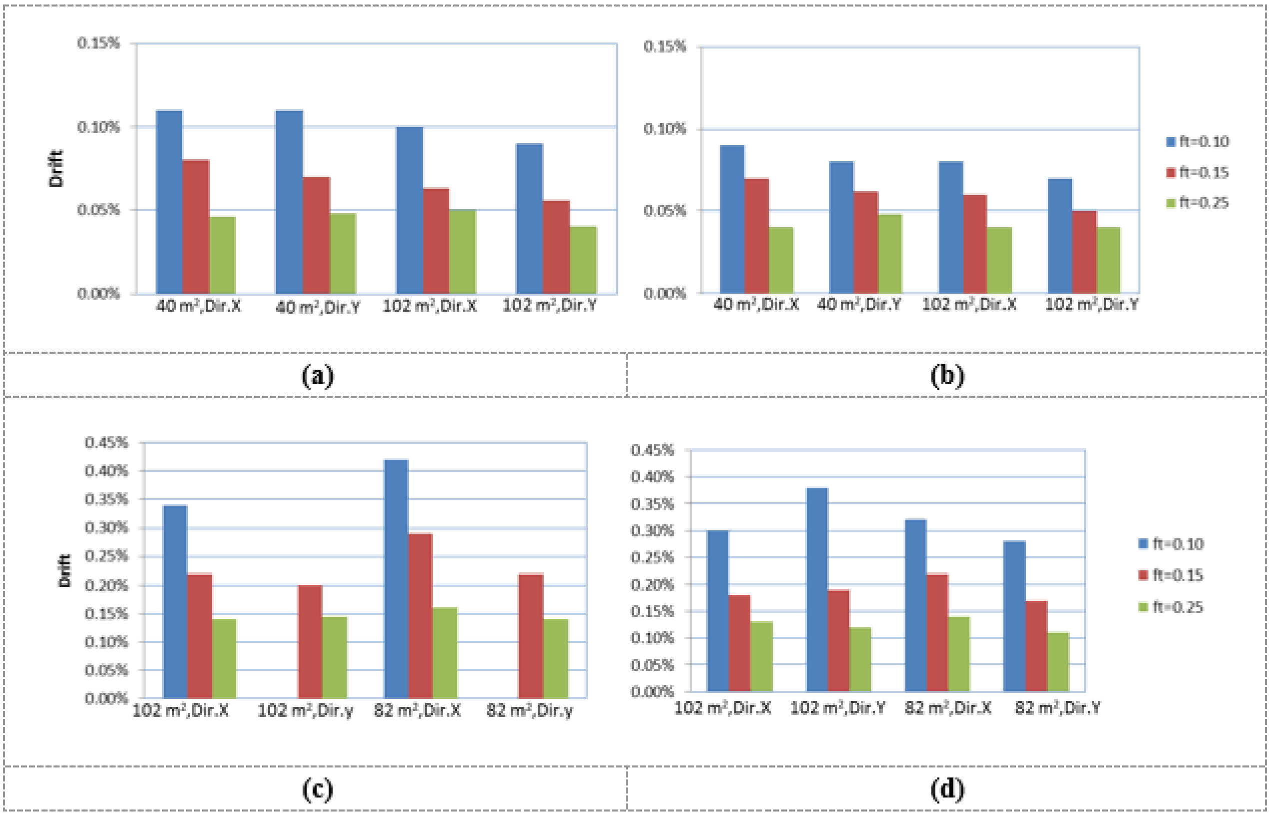


Fig. 15. Drift angle data for prototype models, a) one story with a thickness of walls 22 cm, b) one story with a thickness of walls 35 cm, c) two-story with a thickness of walls 22 cm, and d) two-story with a thickness of walls 35 cm.

failure [1], so the results of this research are presented based on the drift at the first story of the prototype models and maximum drift of the walls. According to the experimental studies three limit states are considered for the CMW with solid clay units [4], Serviceability related to the beginning of inclined cracking, reparability related to inclined cracking fully formed, and safety limit state related to the shear strength of the wall and tie-column crushing which is corresponding to drift angle of 0.15, 0.25, and 0.4%, respectively. The maximum drift of walls in the first story at performance point (intersection point of capacity and demand spectrum in ADRS) and hysteresis damping for various prototype models are presented in Tables 8 and 9. The results are derived based on the performance point in the ADRS format of design Spectra of standard No.2800 at a very high hazard seismic zone (PGA=0.35 g). The performance point is determined in both main direction of 1 and 2-story prototype models with two thicknesses of walls 22 and 35 cm, three tensile strengths of masonry 0.1, 0.15, and 0.25 MPa, and various densities of walls from 5 to 15.5 and from 5 to 9 percent in one and two-story, respectively.

Fig. 15 shows the drift angle in the format of the bar chart. As it is seen the drift angle of masonry walls in one-story confined masonry structures is lower than the serviceability limit state (0.15%) by a large distance, such that buildings with f_t of 0.25 MPa are in the elastic or No-Damage limit state (0.04%). It means that the seismic behavior of these types of structures even with having poor quality of material strength (0.1 MPa) and a minimum 5% density of the walls is at the beginning of the formation of inclined cracking in the walls in the very high seismic zone, whereas the seismic behavior of two-story of confined masonry structures is different from one story models. As it is seen in the structures with high quality of material strength (0.25 MPa), the response of models is near to the serviceability limit state but the drift angle of most models with the material strength of 0.15 MPa is upper than the serviceability and lower than the reparability limit state (0.25%). Only one model which is related to 22 cm thickness of wall and 5% density of wall passes through the reparability limit state. The situation of the structures with the tensile strength of masonry of 0.1 MPa is between the reparability and the safety limit state (0.4%). It means these types of structures going to collapse. In one case which is related to 22 cm thickness of wall and 5% density of wall, the drift is upper than the safety limit state.

To conclude, Table 10 presents the adequacy of the minimum density of walls in each category of prototype models. The one-story prototype models with a minimum 5% density of walls provide the serviceability limit state with the criterion of onset of masonry inclined cracking in the walls and elastic behavior with 0.1 MPa and 0.25 MPa tensile strength of masonry, respectively. The two-story prototype

Table 8. The results of the one-story prototype model.

Thickness of Walls (mm)	Direction	Tensile strength of Masonry (MPpa)	Density of Walls (%)	Drift (%)	Damping (%)	Number of Walls	Area of Plan (m^2)
350	X	0.1	15.50	0.09	8.5	6	40
		0.15	15.50	0.07	5	6	40
		0.25	15.50	0.04	5	6	40
	Y	0.1	9.00	0.08	15.3	3	40
		0.15	9.00	0.06	10.2	3	40
		0.25	9.00	0.05	5	3	40
	X	0.1	8	0.08	7.8	7	102
		0.15	8	0.06	5	7	102
		0.25	8	0.04	5	7	102
	Y	0.1	8	0.07	15.8	6	102
		0.15	8	0.05	12	6	102
		0.25	8	0.04	7	6	102
220	X	0.1	10	0.11	5	6	40
		0.15	10	0.08	5	6	40
		0.25	10	0.05	5	6	40
	Y	0.1	5.50	0.11	14.6	3	40
		0.15	5.50	0.07	13.5	3	40
		0.25	5.50	0.05	10	3	40
	X	0.1	5	0.10	8	7	102
		0.15	5	0.06	7	7	102
		0.25	5	0.05	5	7	102
	Y	0.1	5	0.09	13.7	6	102
		0.15	5	0.06	10.5	6	102
		0.25	5	0.04	8	6	102

models with a minimum 5% density of walls and having a high tensile strength of masonry (0.25 MPa) provide the serviceability limit state with the criterion of the beginning of inclined cracking in the walls, but buildings with having a moderate tensile strength of masonry (0.15 MPa) provide reparability limit state that it means inclined cracking forms in masonry walls in fully formed-shape, buildings with a low tensile strength of masonry (0.1 MPa) don't provide safety limit state and tie-column crushing can occur and masonry walls achieve their shear strength, it means the structure goes to collapse.

According to the definition of Alcocer and his colleagues [4], Teran-Gilmore and his colleagues [7], and Bilgin and Huta [8], the drift limit states of 0.04, 0.15, 0.25 and 0.4% correspond to no damage, immediate occupancy, life safety, and collapse prevention, respectively.

By the results of this study from the point of view of FEMA356 [30] according to Fig. 15 (a and b) and Tables 8 and 10, buildings with one story and minimum 5% density of walls with tensile strengths of masonry 0.1 and 0.15 MPa provide immediate occupancy and for strength of 0.25 MPa, the No-Damage limit state is observed in prototype models. But, according to Fig. 15 (c and d) and Tables 9 and 10, buildings with two-story and minimum 5% density of walls with masonry tensile strength of 0.1 MPa go to collapse prevention, buildings with the strength of 0.15 MPa have life safety performance and buildings with the strength of 0.25 MPa provide Immediate occupancy performance level.

From the point of view of standard No.2800 and considering drift limit of 0.25% corresponding to life safety limit state, according to Fig. 15 (a and b) and Table 10 it seems for one story buildings with having a large distance of drift angle to serviceability limit state, the recommended minimum wall density of 4% for confined masonry buildings in the zone with high and very high hazard (PGA=0.3 and 0.35 g) is adequate and it is expected the building does not suffer any damage such that the structure can be utilized after the earthquake without any need for significant strengthening and repair , but for two- story buildings according to Fig. 15 (c and d) and Table 10 the proposed minimum wall density of 6% for first story is adequate provided that the tensile strength of masonry is upper than 0.15 MPa, such that with masonry tensile strength of 0.15 MPa the building cannot be used after the earthquake without significant repair and strengthening is feasible and for strength of 0.25 MPa the structure can be utilized after the earthquake without any need for significant strengthening and repair, for material with tensile strength of masonry of 0.1 MPa it seems repairing the building is neither possible nor economically reasonable and the structure will have to be demolished after the earthquake, in other word the structure goes to collapse even with having 8% of wall density.

Table 9. The results of the two-story prototype model.

Thickness of Walls (mm)	Direction	Tensile strength of Masonry (MPa)	Density of walls (%)	Drift (%)	Damping (%)	Number of walls	Area of Plan (m^2)
350	X	0.1	8	0.3	18.4	7	102
		0.15	8	0.18	16.7	7	102
		0.25	8	0.13	14.9	7	102
	Y	0.1	8	0.38	25.2	6	102
		0.15	8	0.19	24	6	102
		0.25	8	0.12	20.5	6	102
	X	0.1	8	0.32	21.4	6	82
		0.15	8	0.22	20	6	82
		0.25	8	0.14	16.8	6	82
	Y	0.1	9	0.28	25.6	5	82
		0.15	9	0.17	23.4	5	82
		0.25	9	0.11	20	5	82
220	X	0.1	5	0.34	20.4	7	102
		0.15	5	0.22	19	7	102
		0.25	5	0.14	16.3	7	102
	Y	0.1	5	Not Converged (Failed)	-	6	102
		0.15	5	0.2	23.5	6	102
		0.25	5	0.14	19.7	6	102
	X	0.1	5	0.42	24	6	82
		0.15	5	0.27	23.2	6	82
		0.25	5	0.16	20.7	6	82
	Y	0.1	6	Not Converged (Failed)	-	5	82
		0.15	6	0.22	23	5	82
		0.25	6	0.14	20	5	82

Table 10. Adequacy of the minimum density of masonry walls.

No. of Stories	Wall thickness (cm)	Wall density (%)	Tensile strength (MPa)	Serviceability	Reparability	Safety
1	22	5	0.1	✓	✓	✓
			0.15	✓	✓	✓
			0.25	✓	✓	✓
	35	8	0.1	✓	✓	✓
			0.15	✓	✓	✓
			0.25	✓	✓	✓
2	22	5	0.1	×	×	×
			0.15	×	×	✓
			0.25	×	✓	✓
	22	6	0.1	×	×	×
			0.15	×	✓	✓
			0.25	✓	✓	✓
	35	8	0.1	×	×	✓
			0.15	×	✓	✓
			0.25	✓	✓	✓

✓: Adequate; ×: Not Adequate

5- Conclusions

In this research, the value of density of confined masonry walls with a thickness of 22 and 35 cm based on three performance levels is studied numerically. Three common plans with assuming shear failure type of walls in one and two-story and masonry tensile strength of 0.1, 0.15, and 0.25 MPa are considered. In most codes this value is recommended without taking into account mechanical and geometrical parameters of the structure, in this research, the effective parameters are applied and the results in the form of drift angle in the masonry walls are compared with the three limit states serviceability (immediate occupancy), reparability (life safety) and safety (collapse prevention) corresponding to the drift limit of 0.15, 0.25, and 0.4%, respectively. The evaluation was conducted based on the capacity spectrum method of ATC-40 and the design Spectra of standard No.2800 in a very high hazard seismic zone.

The performance level of one-story confined masonry structures with at least 5% wall density and 0.1 MPa tensile strength of masonry is serviceability.

The performance level of two-story confined masonry structures with at least 5% wall density and 0.25 MPa tensile strength of masonry is serviceability.

The performance level of two-story confined masonry structures with at least 5% wall density and 0.15 MPa tensile strength of masonry is reparable.

The performance level of two-story confined masonry structures with at least 5% wall density and 0.1 MPa tensile strength of masonry is safety.

From the point of view of standard No. 2800 and considering Life safety as a performance level, one-story buildings with the recommended minimum wall density of 4% in the zone with high and very high hazard (PGA=0.3 and 0.35g) perform adequately and it is expected the building does not suffer any damage such that the structure can be utilized after the earthquake without any need for significant strengthening and repair, the adequacy of proposed minimum wall density of 6% for two-story of confined masonry buildings depends on tensile strength of masonry buildings, such that the building with masonry tensile strength more than 0.15 MPa provide life safety performance level and for more than 0.25 MPa the structure can be utilized after the earthquake without any need for significant strengthening and repair, but for material with a tensile strength of masonry 0.1 MPa it seems the structure goes to collapse.

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HOW TO CITE THIS ARTICLE

F. Ranjbaran, M.R. Kianibakhsh, Seismic Performance Assessment of Confined Masonry Buildings Based on Displacement, AUT J. Civil Eng., 5(2) (2021) 225-244.

DOI: 10.22060/ajce.2021.18224.5666