Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1652

# A SIMPLIFIED BEHAVIORAL MODEL FOR NONLINEAR SEISMIC ANALYSIS OF CONFINED MASONRY WALLS

Fariman Ranjbaran[1] and Mahmood Hosseini[2]

## ABSTRACT

Using confined masonry buildings is very common in many parts of the world such as Europe, Asia, and Latin America as governmental and private buildings. Past earthquakes have shown the vulnerability of this type of buildings in several cases, however, experimental results represent ductile behavior of structural elements of such buildings, i.e. confined masonry walls. Design of buildings of this type is usually performed in a prescriptive manner without numerical modeling, and out of the framework of an analysis-designing process. The reasons behind this fact are firstly the lack of sufficient knowledge concerning the exact behavior and performance of such structural elements against the lateral forces, and secondly very time-consuming analysis procedures. The purpose of this research is to identify the major factors affecting the behavior of the confined walls against lateral and vertical forces, and then to present a simplified behavioral model precise, and at the same time, simple enough to be used by professional engineers. The proposed model can show the wall behavior before and after cracking. The required analyses have been performed by using DIANA (9.3) software, which is a powerful tool for numerical modeling of such structural elements. A series of nonlinear static analyses in parametric form has been performed using a wide range of effective factors on confined masonry walls with or without opening. Then, based on the numerical results, some simple formulas have been proposed to express the relationships between the lateral strength of the confined wall and the wall specifications, including the initial stiffness, the secondary stiffness after cracking, the ultimate strength, and ductility, to be used in engineering programs such as SAP, which are widely used in engineering firms, by practicing engineers.

## Introduction

Masonry buildings with confined brick walls can be found enormously in many parts of the world, and their performance in past earthquakes has been relatively good. However, many

[1]Ph.D. Student, Earthquake Eng. Dept., School of Engineering, Science and Research Branch of the Islamic Azad University (IAU), and Instructor, Civil Eng. Dept., Islamshahr Branch of the Islamic Azad University, Islamshahr, Iran, Email: ranjbaran_far@iiau.ac.ir
[2]Associate Professor, Structural Eng. Research Center, the Int'l Institute of Earthquake Engineering and Seismology (IIEES), Tehran, Iran, Email: hosseini@iiees.ac.ir

of these buildings have shown unsatisfactory response in recent earthquakes, and this shows that there are still some shortcoming(s) in their seismic design. Therefore, it is necessary to evaluate their seismic behavior more thoroughly. On the other hand, for seismic analysis of this type of buildings with high precision either advanced finite element analyses or distinct element analyses are required, which are very time consuming. Therefore, a simplified method which is less time consuming and has acceptable precision is very useful and acknowledged, particularly by practicing engineers for seismic assessment of existing buildings for proposing their retrofit design. Stiffness, ductility, and ultimate strength are three main design parameter of the wall. These parameters depend on the strengths of bricks and mortar, the mortar thickness, dimensions of the wall and their ratios, the number and dimension of openings, if any, and their size ratio to the wall dimensions in horizontal and vertical directions, and finally the vertical loads on the wall. Non-homogeneity of the wall materials has made its modeling very difficult, particularly when post-cracking analysis is concerned, and none of the proposed models is quite satisfactory in all aspects.

In past investigation two points are considerable in the presented analytical models. First, the presented models are complicated and need spending much time and expenses, and second, the simplified models do not take into account fully the effective parameters in relation with the behavior of the element. Therefore in this research these two mentioned points are considered together. It is tried to present an analytical model in case of confined masonry walls having enough precision and at the same time being not complicated. On this basis, in this paper by performing a series of very detailed analyses on several samples of confined walls with or without opening(s) and having various confinements and different geometrical features and mechanical characteristics, a simplified analysis model is proposed in which the interaction of wall and its confining ties, and the post-cracking behavior of the wall are taken into consideration. In the detailed analyses, performed by DIANA (version 9.3) computer program, Lourenco model (Lourenco 1996) has been used for the nonlinear behavior of wall, which takes into account the different strength in directions parallel and perpendicular to bricks interfaces with mortar. More than 100 detailed models have been considered for analyses, in which cracking in concrete, interaction between ties and wall, and interaction between walls and its foundation, reinforcing steel bars in ties and orthotropic behavior in masonry wall have been considered. Three different values from low to high have been considered for the wall length, and also for the amount of vertical load on the wall, and two values for the wall thickness, to see how these parameters affect the wall seismic behavior. Based on the force displacement curves obtained by detailed analyses the simple relationships between the seismic input parameters and the wall specifications have been obtained, which include the initial stiffness, the secondary stiffness after cracking, the ultimate strength, and the ductility. These formulas are very suitable to be used in engineering programs such as SAP, which are widely used in engineering firms.

## Definition of the Problem

It can be realized in available codes or guidelines for seismic design and evaluation of the confined masonry buildings that their design are generally very conservative, and is mostly based on the response or reaction in the elastic zone. This is while the behavior of confined masonry walls can be ductile, and this fact may be included in some performance levels and consequent nonlinear analyses (Ruiz-Garcia and Negrete 2008). One of the approaches of such analysis is the static nonlinear analysis method or Push Over Analysis (POA). The purpose of this research is proposing a simplified analytical model for the determining the seismic capacity of confined masonry walls. In this model, determination of 5 indices, including initial stiffness, initial crack capacity, ultimate capacity, ductility value related to final capacity and finally residual resistance are assumed as functions of effective factors. These factors include wall thickness, tensional and compressional strengths of masonry unit, surcharge, dimensional ratio of wall and dimensions of opening, if any. Using DIANA (version 9.3) software, which has the capability of numerical modeling such structural elements considering their behavior up to the facture stage, it was tried to conduct a series of POA in parametric form on confined masonry walls of clay bricks with or without opening.

Two values of 22cm and 35cm were considered for the wall thickness, which are common thickness values in masonry walls. Tensional strength of masonry unit (perpendicular of bed joint was considered to be tensional strength of mortar, varying from 0.03 to 0.25Mpa, and compressional strength was chosen proportional to the tensional strength. The wall height was assumed to be 3 m and its length 2, 3, and 4m. The amount of surcharge was considered to be 11.6, 23.2, and 34.8N/mm. The ratio of opening length to the wall length was assumed to be for the 2m long wall 0.125, 0.25, and 0.375 and for the 3m and 4m long walls 0.25, 0.5, and 0.75, and the ratio of opening height to the wall height was considered to be 0.17, 0.33, and 0.5. Ties were considered base on the recommendations of National Iranian Code of Practice for Seismic Design of Buildings (Standard No. 2800), and their mechanical properties were assumed to be the same for all cases. Horizontal and vertical ties were modeled in the form of reinforced concrete beams with dimensions of 20×20cm for vertical ties, and 20×20 and 20×35cm for horizontal ties, corresponding to 22 and 35cm walls respectively. Reinforcement inside ties was assumed to be consisted of 4 steel bars of 10mm diameter with the yielding strength of 300Mpa. Compression strength of concrete was also considered to be 15Mpa.

Using the aforementioned quantities and taking into account the wall weight in loading process, and assuming the boundary conditions of wall as a cantilever, the POA in displacement control state were performed with a target displacement of 30mm (0.01h) (Moroni et al. 1994), and the capacity curved for all of the numerical models were developed. Eventually, a cluster of curves were created for each of the aforementioned parameters. Using these curves some simple formulas were developed for predicting the five aforementioned indices based on the effective parameters, which are similar to FFMA 273 formulas, but have more precision.

## Modeling

For modeling the masonry walls in DIANA (version 9.3) software, the Continuum Finite Element method (macro-model method) was used, which has relatively high precision and is also more appropriate for studying the general behavior of wall. In this method, masonry wall is simulated in the form of a continuous homogenized environment and Rankin's Model and Hill's Model are used for expressing the inelastic behavior in tension and in compression respectively. These models also take into account the orthotropic behavior. It must be noted that amounts of fracture energy in compression and tension are considered to be the common values in clay brick materials. Furthermore, the combined crack-shear-crush interaction model is believed to be suitable for the simulating fracture caused by tension, fractional slide caused by shear and crush caused by compression. Regarding the existence of friction between wall and foundation after cracking, this model is also considered to be suitable for simulating wall-foundation interaction (Mohebkhah et al. 2008). In wall-ties interaction normal and shear stresses are functions of total relative displacement, i.e. width and slide of crack. Tension and shear strengths and stiffness after crack between tie and wall are neglected and behavior of element is considered in brittle type without softening. For avoiding penetration of wall and tie elements into each other, a big number is considered for the axial stiffness in compression. For the numerical stability of interaction element, sliding, with initial shear stiffness, is likely. For modeling the concrete material of ties the Total Strain Rotating Crack is used (Hashemi and Mosalam 2007). Stress-strain relation defined in modeling for tensional stress is Maekawa model, and for compressional stress is the perfect elastoplastic considering 28-day compression strength of concrete. Finally, for modeling the reinforcement of ties the longitudinal bars are assumed to have full bond with concrete around them and follow von Mises Criteria with perfect elastoplastic flow criteria.

## Verification of Numerical Modeling

For verifying the numerical modeling, the numerical models developed based on the behaviors explained in the previous section are compared with some experimental models. These models include a double-storey concrete frame with infill walls for verifying the ties modeling (Vecchio and Basil Emara 1992), one masonry wall without confinement for verifying masonry wall modeling (Tomazevic and Klemenc 1997), and also two confined masonry walls for verifying the modeling of the compound system of masonry wall and ties (Tomazevic and Klemenc 1997; Marinilli and Castilla 2004). Fig. 1 shows comparison of experimental results with those obtained by the developed numerical models in this study.

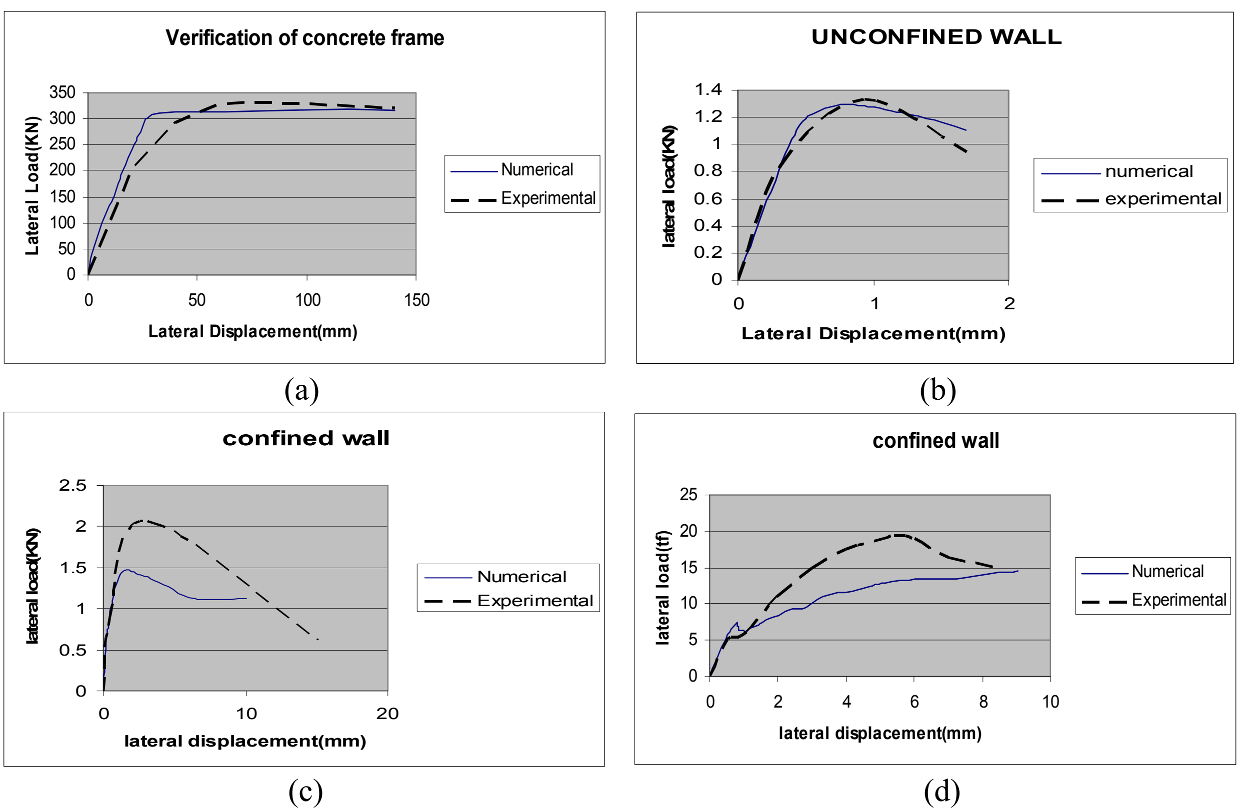


Figure 1. Comparison between experimental and numerical models: (a) related to the double-storey concrete frame with infill walls, (b) related to masonry wall without confinement, and (c) and (d) related to confined masonry walls

It is seen in Fig. 1 that in cases of the double-storey concrete frame with infill walls and the masonry wall without confinement there are good agreement between the numerical and experimental results, however, for cases of confined masonry walls the numerical results underestimate the experimental results by a ratio of about 1.4. This difference is believed to be due to the interaction between ties and wall, which can not be modeled quite realistically by the numerical model. On this basis a coefficient of 1.4 is considered as the calibration coefficient in this study.

## Simplified Analytical Models

After verification of numerical modeling the simplified analytical models can be developed by considering some sets of values for the five main aforementioned factors, affecting the behavior of the confined masonry walls. The mathematical forms of the simplified analytical models are selected based on the basic relationships in mechanics of structures. The values of the used factors or coefficients appearing in the considered mathematical formulas are obtained by regression analysis through the least squares method. A sample of the curved fitted to one set of plotted numerical results is shown in Fig. 2.

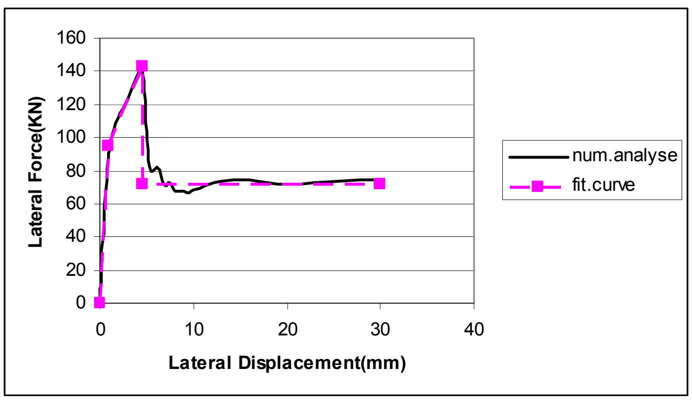


Figure 2. A sample of the lateral force-displacement curves of confined walls obtained by numerical analyses and the fitted curve for developing the simplified analytical formulas

It is seen in Fig. 2 that the fitted curve is consisted of four specific linear segments, of which the initial stiffness, the yielding displacement and secondary stiffness, the maximum strength and initial displacement ductility, the strength drop after reaching the its maximum value, and finally, the maximum ductility (if achievable) can be obtained. These specifications are discussed in more detail in the following parts of this section.

### Initial Stiffness

Based on the prevalent relations for flexural and shear stiffness of a deep beam the stiffness coefficient values of confined walls, can be expressed by Eqs. (1) and (2), respectively, for walls without opening and walls with opening.

$$K = \frac{1}{\left[\dfrac{h^3}{a \times 3 \times E \times I_w} + \dfrac{bh}{G \times A_w}\right]} \qquad (1)$$

$$K = \frac{1}{a \times \left[ b^{\left(\frac{l_o}{l_w} \times \frac{h_o}{h_w}\right)} \right] \times \left[ \frac{h^3}{c \times 3 \times E \times I_w} + \frac{dh}{G \times A_w} \right]} \tag{2}$$

In Eqs. (1) and (2) parameters E and G are respectively modulus of elasticity and shear modulus of the wall materials, $I_w$ and $A_w$ are moment of inertia and cross-sectional area of the horizontal section of the wall, h and $h_w$ are the wall height, $l_w$ is the wall length, $h_o$ and $l_o$ are the opening height and length, and a, b, c and d are parameters whose values depend on the wall specifications, and are given in Table 1.

Table 1. 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 | 29.46 | 32.6 |
| b | 1.77 | 237 | 0.43 | 0.55 | 0.45 | 0.49 | - | - | - | - | 0.98 | 0.95 | 0.064 | 0.067 | -5.55 | -5.64 |
| c | - | 2.79 | 0.55 | 0.83 | 0.79 | 0.88 | - | - | - | - | 18.64 | 53.58 | 1.23 | 1.28 | -4.31 | -7.83 |
| d | - | 1.65 | - | - | 0.33 | 0.28 | - | - | - | - | - | | 0.91 | 0.91 | 4.88 | 8.45 |
| $R^2$ | 0.97 | 0.91 | 0.94 | 0.98 | 0.86 | 0.92 | 0.85 | 0.93 | 0.85 | 0.88 | 0.98 | 0.98 | 0.95 | 0.89 | 0.90 | 0.90 |

*Related to wall with opening

The values of correlation coefficient, R, in regression analysis, shown in Table 1, show the high precision of the proposed formulas. An important point with regard to the fracture mechanism of wall is the effect of tensional strength of masonry, as discussed hereinafter.

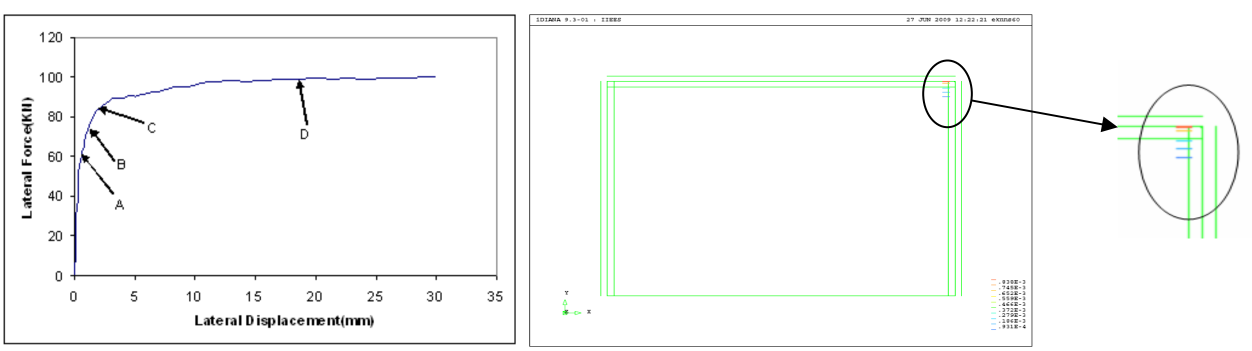


Figure 3. Capacity curve (left), and beginning of cracking at top of the vertical tie (right)

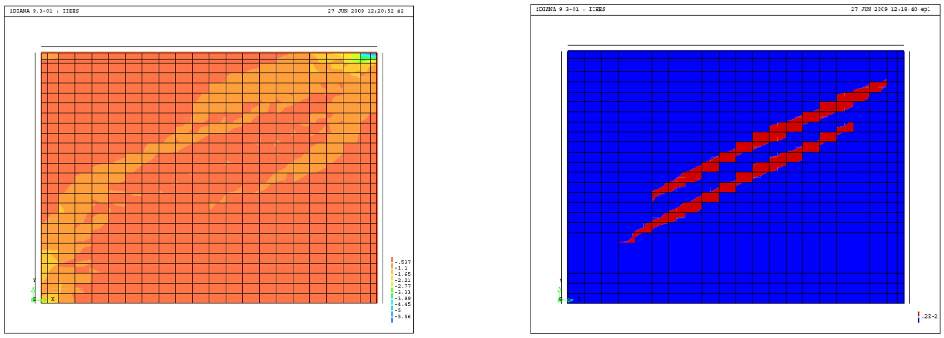

Figure 4. Distribution of principal compressional stress (left) and opening and development of cracks (right)

In Fig. 3, the load-displacement or the capacity curve of wall with a lower tensional strength ($f_t$=0.03Mpa), and the beginning of cracking at top of the vertical tie are shown. Also in Fig. 4 the distribution of principal compressional stress, and opening and development of cracks in the wall are shown. In Fig. 3 point A in the capacity curve is corresponding to start of cracking in the tie (shown in figure at right), point B is corresponding to the start of diagonal cracking in the wall, Point C is corresponding to widening of cracks, as shown in Fig. 4 (right), and point D is corresponding to the ultimate strength of wall ($f_{cy}$=5.47Mpa). It can be seen in Fig. 3 that the capacity curve of the confined wall with very low tensional strength is very similar to that of a concrete frames with infill panel (see Fig 1-a). For higher values of tensional strength, as shown in Fig. 5, the capacity curve becomes more and more similar to that shown in Fig. 2. It can be said that after cracks in the wall start widening the strength of the confined wall drops suddenly and reaches the strength of the infilled frame.

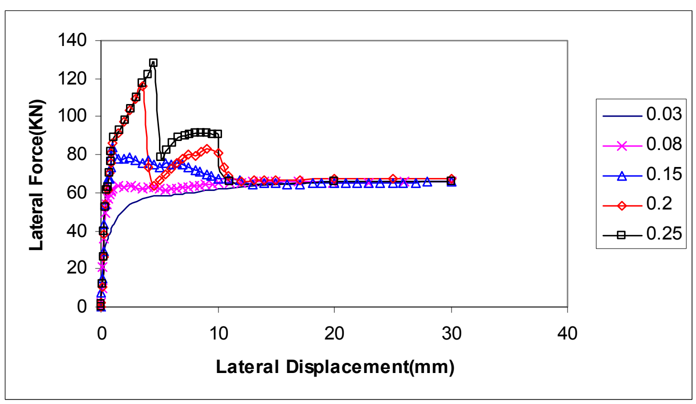


Figure 5. Affect of tensile strength in behavior of wall

## Maximum Strength

For the maximum or ultimate strength, $Q_u$, based on the curves fitted to the numerical results, Eqs. (3) and (4), respectively for wall without opening and walls with opening, can be introduced, getting idea from equation (7-5) in FEMA 273.

$$Q_u = a \times 1.4 \times \left[ f_t \times A_w \times \frac{l_w}{h_w} \right]^b \times \left[ 1 + \frac{f_a}{f_t} \right]^c \tag{3}$$

$$Q_u = a \times 1.4 \times \left[ f_t \times A_w \times \frac{l_w}{h_w} \right]^b \times \left[ 1 + \frac{f_a}{f_t} \right]^c \times \left[ d^{\left( \frac{l_o}{l_w} + \frac{h_o}{h_w} \right)} \right] \tag{4}$$

In Eqs. (3) and (4) the basic parameters are the same as those introduced in Eqs. (1) and (2), and $f_t$ and $f_a$ are the tensional strength of the wall material and the compressional stress created in the wall because of the surcharge, and $l_p$ and $h_p$, are the length and height of piers (the part of wall locating at either side of an opening).

**Yielding or Elastic Limit Strength**

Numerical results show that the wall without opening reaches its yielding strength with start of cracking in ties, and this yielding strength is well below the maximum strength of the wall. However, the wall with opening reaches its yielding strength with formation of plastic hinges at both ends of piers, and its yielding strength is approximately the same as its maximum or ultimate strength, since the existence of opening gives a frame-like behavior to the wall. The amount of this strength, which can be called the elastic limit of the wall, is obtained based on the curve fitted to force – displacement curve resulted from numerical analyses. Using regression analysis, the following simple formula can be suggested for the yielding strength of the wall.

$$Q_p = a \times Q_u \tag{5}$$

The value of parameter *a* is given in Table 1, and as it can be seen, it is very close to unity for walls with opening. Also for walls without opening the values of this parameter is very close to the values of 0.7 to 0.8 given by Tomazevic analytical formula.

**Residual Strength**

After reaching its ultimate strength, the wall shows a reduced or residual strength which maintains almost constant with increase in the wall top displacement, until reaching the ultimate displacement. Existence of the residual strength and the capacity of considerable displacement is mainly because of the existence of ties and their protecting effect on the wall integrity, and to some extent due to the friction existing in the crack area of wall. Based on regression analyses Eqs. (6) and (7) have been obtained for walls without opening and walls with opening respectively, which give the residual strength as a function of length to height ratio and amount of surcharge and also tensional strength of masonry unit.

$$Q_r = a \times \left[ \frac{l_w}{h_w} \right]^b \times c^{f_a} \tag{6}$$

$$Q_r = \exp\left[ a \times \left( \frac{l_w}{h_w} \right)^b \times c^{f_a} \times d^{\left( \frac{l_o}{l_w} + \frac{h_o}{h_w} \right)} \right] \tag{7}$$

**Displacement Ductility**

Referring to Fig. 5, displacement ductility can be defined as ratio of the displacement value at the maximum or ultimate strength to displacement value at the initial strength. Considering that these strengths values are dependent on the amounts of vertical loading or surcharge on the wall, length to height ratio, and tensional and compressional strengths of masonry materials, a single formula can not be given to cover all cases. For walls without opening, based on regression analyses, Eq. (8) can be introduced.

$$D = [a \times f_t] + \left[ b \times \left( \frac{l_w}{h_w} \right) \right] + \left[ c \times \left( \frac{f_a}{f_t} \right) \right] + d \tag{8}$$

For walls with opening, since the wall thickness is an effective factor in the amount of compressional stress in the wall Eqs. (9) and (10) are given respectively for walls of 22 cm thickness and walls of 35 cm thickness.

$$D = \begin{cases} 0.6 \times [5.88 - (24.7 \times f_a) & \frac{l_p}{h_p} > 1 \\ 5.88 - (24.7 \times f_a) & 0.75 \le \frac{l_p}{h_p} \le 1 \quad ; R^2 = 0.94 \\ 1.3 \times [5.88 - (24.7) \times f_a] & \frac{l_p}{h_p} < 0.75 \end{cases} \tag{9}$$

$$D = \begin{cases} 0.68 \times [5.68 - (31.32 \times f_a) & \frac{l_p}{h_p} > 1 \\ 5.68 - (31.32 \times f_a) & 0.75 \le \frac{l_p}{h_p} \le 1 \quad ; R^2 = 0.97 \\ 1.8 \times [5.68 - (31.32) \times f_a] & \frac{l_p}{h_p} < 0.75 \end{cases} \tag{10}$$

Difference between the formulas related to walls with opening of 22 cm thickness and wall of 35 cm thickness is basically due to the difference between the behaviors of these walls, which itself because of remarkably different values of compressional stresses in walls with opening, having different thicknesses. This difference is not so much in case of wall without opening. It may be seen in Fig. 5 that with increase in amount of surcharge and also amount of length to height ratio the wall ductility decreases, in case of walls without opening, while this decrease is not observed in case of wall with opening.

## Conclusions

In this research, using of DIANA Software (9.3), it was tried first to perform a set of nonlinear static analyses on confined masonry walls with and without opening. These analyses were preformed in the form of sensitivity analysis of the numerical models versus the known effective factors on the behavior of confined masonry walls. Then based on the results of numerical analyses, some simplified relationships for lateral force – displacement behavior were developed. In the proposed simplified formulas the effects of geometrical factors of walls as well as the mechanical specifications of the wall materials and also the amount of the existing surcharge are taken into account. The relatively high values of coefficient of correlation for the parameters used in the proposed simplified formulas show their relatively high precision. To use the simplified formulas in engineering practice for evaluation of buildings with confined walls, some simple elements, like plastic shear hinge, can be used in the conventional software such as SAP or ETABS. The specifications of these plastic shear hinges can be easily determined based on adjusting their values with the corresponding values in the proposed formulas.

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