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A PARAMETRIC STUDY ON UNREINFORCED AND CONFINED MASONRY WALL BEHAVIOR

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ABSTRACT

Although it is a known fact that confined masonry construction has many advantages over unreinforced masonry construction, it has never been popular in Turkish construction practice. One of the reasons for this issue is that the seismic regulations in Turkey do not explicitly enforce the use of confined masonry structural systems. Now, being on the verge of releasing a new version of the Turkish seismic code, it seems to be an appropriate time to adapt confined masonry to the Turkish construction practice. This study intents to reveal why such attempts are necessary by comparing the behaviour of unreinforced and confined masonry on the component (wall) scale. In the first phase of the study, a rigorous literature survey is conducted on the available laboratory tests regarding unreinforced and confined masonry walls. Then idealized tri-linear capacity curves are proposed following the recommendations of Tomazevic (1999) with three performance points (i.e. cracking, maximum and ultimate). The empirical formulations in literature related to these performance points are investigated and the ones with closest match to the experimental behaviour are selected to represent the idealized behaviour of unreinforced and confined masonry walls. After verifying that the selected set of performance parameters for unreinforced and confined masonry walls reasonably match with the experimental results, this idealized analytical information is used to conduct a parametric study for the comparison of the behaviour of these two different types of masonry walls. The parameters used in this analytical study are the basic geometrical and mechanical properties of masonry walls. The results of the parametric study reveal the superior behaviour of confined masonry walls over unreinforced masonry walls.

INTRODUCTION

Nowadays masonry construction is not in well deserved place because of the tendency to construct reinforced concrete structures instead. For many people, masonry is just an historical, rural or non-engineered construction type. However it should not be disregarded that masonry has many advantages over reinforced concrete structures for low rise residential building construction as low cost, durability, material availability, thermal isolation, fire resistance and low maintenance. It requires little technology and skill and this provides masonry buildings to be constructed without an engineering touch.

Although construction of new masonry structures is not popular, there still exists a considerable percent of masonry buildings in the building stocks of many earthquake prone countries including Turkey. On the verge of releasing new national seismic code and standards in Turkey, which also includes the seismic design of new and assessment of existing masonry structures, this study is an

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attempt to exhibit the superiority of confined masonry construction over traditional unreinforced masonry construction although the former construction type is not explicitly enforced by the previous or the current code of practice in Turkey. However it is still possible to develop simple empirical design guidelines for confined masonry construction just like unreinforced masonry construction. In this respect, this study compares the seismic performances of unreinforced and confined masonry walls by using an idealized approach that is based on the experimental behaviours of these two masonry wall types.

In the initial phase of the study, the experimental work related with unreinforced and confined masonry walls has been investigated thoroughly. The scope of this study is limited to monotonic and cyclic tests on regular unreinforced and confined masonry walls with no horizontal or vertical reinforcement, multiple tie columns, openings in walls, special construction features and/or interventions applied to the walls. Then idealized capacity curves are proposed for unreinforced and confined masonry walls based on the considered experimental data. Capacity curve parameters for trilinear idealization are selected from the analytical and empirical equations given in the literature. Then the proposed capacity curves are verified by using the experimental results. In the last phase, a parametric study is conducted by using the idealized capacity curves of these two masonry wall types in order to compare their seismic performance for different geometrical and material properties and loading conditions.

PAST EXPERIMENTAL RESEARCH ON MASONRY WALLS

A large number of experimental studies regarding the behaviour of unreinforced and confined masonry walls conducted by many different researchers have been investigated in the first phase of this research. The details of this literature survey can be found elsewhere (Erkoseoglu 2014). The experimental studies that are not compatible with the scope of this research and that possess missing/unreliable data and abnormal experimental results have been eliminated and finally 20 unreinforced and 23 confined masonry wall tests are selected to construct the experimental database used in this study. The names of the wall specimens used in this study are provided with their original names and references in Tables 1-2. In the case of cyclic loading, each loading direction is considered separately as shown with minus and plus signs in the original names of the specimens. The wall tests in the considered experimental database possess a significant variation in geometrical dimensions, material properties, loading conditions and experimental setup. Detailed information about the properties of wall tests is provided in Erkoseoglu (2014).

Table 1. Reference list for unreinforced masonry walls used in this study

Wall name (this study)	Original wall name	Reference
URM-W1	B1	Tomazevic and Klemenc (1997)
URM-W2	B2	
URM-W3	B3	
URM-W4-1	7.1a.2	ESECMASE (2007)
URM-W5	7.1a.3	
URM-W6-1	7.1a.8	
URM-W7	7.1a.1 (-)	
URM-W4-2	7.1a.2 (-)	
URM-W8	7.1a.6 (-)	
URM-W6-2	7.1a.8 (-)	
URM-W9	3D-L0-H0VO-48-1	Yoshimura et al. (2004)
URM-W10	2D-L0-H0VO-84-1	
URM-W11	3D-L0-H0VO-84-1	
URM-W12-1	CL04 (+)	Magenes et al. (2008)
URM-W12-2	CL04 (-)	
URM-W13-1	CL05 (+)	
URM-W13-2	CL05 (-)	
URM-W14-1	CL07 (+)	
URM-W14-2	CL07 (-)	
URM-W15	CL09 (+)	

Table 2. Reference list for confined masonry walls used in this study

Wall name (this study)	Original wall name	Reference
CM-W1	A1	Tomazevic and Klemenc (1997)
CM-W2	A2	
CM-W3	A3	
CM-W4	M0	Aguilar and Meli (1996)
CM-W5-1	M1	Perez-Gavilan and Manzano (2012)
CM-W5-2	M2	
CM-W6	1-1(+) (concrete)	Yanez et al. (2004)
CM-W7	1-2(+) (concrete)	
CM-W8-1	1-1(+) (clay)	
CM-W8-2	1-2(-) (clay)	
CM-W9-1	1-1(+) (clay)	
CM-W9-2	1-2(-) (clay)	
CM-W10	W2.4.	Gouveia and Lourenço (2007)
CM-W11	4-HOVO	Yoshimura and Kikuchi (1996)
CM-W12	CM30J-1	Bourzam et al. (2008)
CM-W13	CM30J-2	
CM-W14-1	M1 (+)	Marinilli and Castilli (2004)
CM-W14-2	M1 (-)	
CM-W15	2D-L1-H0VO-48-1	Yoshimura et al. (2004)
CM-W16	3D-L1-H0VO-48-1	Yoshimura et al. (2004)
CM-W17	2D-L1-H0VO-84-1	Yoshimura et al. (2004)
CM-W18	2D-H1-H0VO-84-2	Yoshimura et al. (2004)
CM-W19	3D-L1-H0VO-84-1	Yoshimura et al. (2004)

IDEALIZATION OF SEISMIC PERFORMANCE OF MASONRY WALLS

Seismic performance of masonry walls is too complex and depends on many factors like geometrical properties, mechanical characteristics and loading type and intensity. Therefore it is a challenging task to idealize this performance by using a simplified approach with some reasonable accuracy. In this study, the considered experimental database is employed in order to develop such a simple tool. In other words, an idealized analytical model to simulate the resistance of unreinforced and confined masonry walls is proposed that has the closest match with the experimental results. The first step to achieve this task is to decide on the shape of the resistance envelope. Examining the considered experimental data, it can be observed that a piece-wise linear idealization with a trilinear form matches well with the experimental behaviour in the whole range of response (Figure 1). This was also stated by Tomazevic (1999), who proposed both bilinear and trilinear approximations for the seismic resistance of masonry wall components. Hence, a trilinear idealization is considered in this study to represent the seismic performance of masonry walls. In a trilinear approximation, there exist three performance points or limit states (Figure 2). First limit state is the elastic limit state where first significant cracking takes place in a masonry wall by definition. Exceeding the elastic limit induces an abrupt change in the stiffness of the wall. Second limit state is maximum resistance which is significantly influenced by different wall parameters and the loading protocol. Third limit state is ultimate state where the wall reaches to its maximum displacement just before a significant loss of capacity or even collapse. Three pairs of values in terms of force and displacement are sufficient to define the idealized capacity curve. These parameters, which are presented in Figure 2, are referred as limit state parameters in this study.

CONSTRUCTION OF IDEALIZED CAPACITY CURVES

The idealized capacity curves for unreinforced and confined masonry walls can be constructed by determining the six limit state parameters in terms of force and displacement. To achieve this task, available formulations in the literature are investigated and the ones that yield the best result in comparison with the experimental behaviour are selected. Capacity curve construction procedure is similar for both unreinforced and confined masonry walls but different formulations are used for the idealization as discussed below.

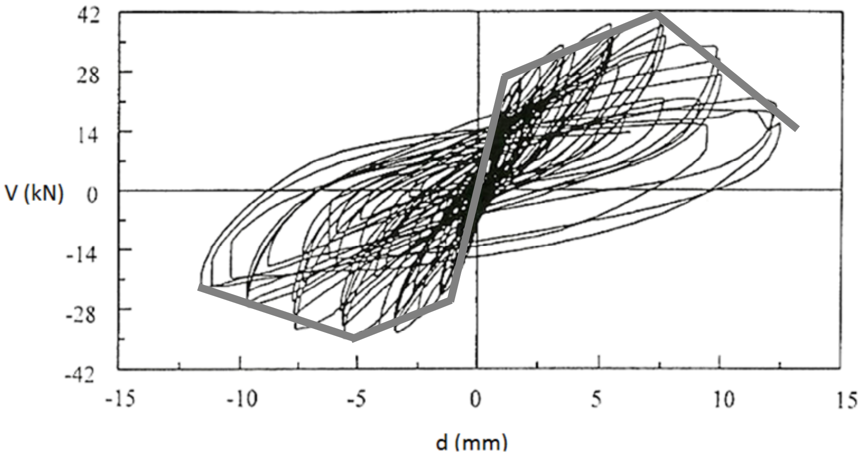

Figure 1. Typical hysteresis curve obtained during the cyclic test of a masonry wall (Tomazevic, 1999) and the idealized trilinear envelope

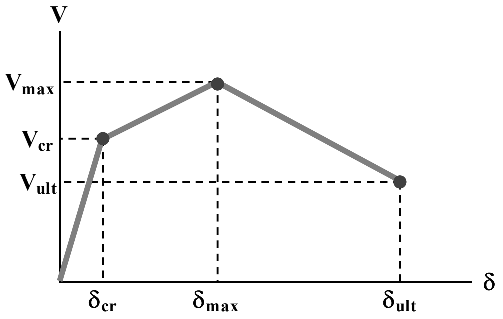

Figure 2. Idealization of experimental hysteresis envelope with trilinear relationship

For unreinforced masonry walls, the idealization procedure starts with predicting the maximum resistance (i.e. V_{max}). It is a known fact that V_{max} depends on many factors like aspect ratio, axial load ratio, strength of the wall, etc. The governing behaviour mode during maximum resistance should also be determined. Tomazevic (1999) proposed a set of formulations to represent the maximum resistance due to diagonal tension (V_{dt}), sliding shear (V_{ss}) and flexure (V_{fl}) modes of unreinforced masonry walls.

$$V_{ds} = A_W \frac{f_t}{b}\sqrt{\frac{\sigma_y}{f_t}+1} \tag{1}$$

$$V_{ss} = A_W \, (f_{vo} + \mu \sigma_y) \tag{2}$$

$$V_{fl} = \frac{M_{ru}}{\alpha h} \tag{3}$$

In these formulations, A_w, h, α and b stand for cross-sectional area of the wall, height of the wall, constraint parameter and shear stress distribution factor; f_t, μ, σ_y and f_{vo} represent tensile strength of masonry, coefficient of friction between masonry unit and mortar, vertical stress on the wall and shear bond strength at zero compression, respectively. Parameter M_{ru} denotes the flexural resistance moment of the wall and can be further defined as

$$M_{ru} = \frac{\sigma_y L^2 t}{2}\left(1 - \frac{\sigma_y}{f_m}\right) \tag{4}$$

where L, t and f_m stand for wall length, wall thickness and compressive strength of masonry, respectively. Among these behaviour modes, it is assumed that the one with the minimum capacity governs V_{max}.

After determination of V_{max}, resistance at cracking (V_{cr}) is obtained by using the below formulation (see Equation 5). Parameter C_{cr} in this equation was proposed by Tomazevic (1999) and it is empirically obtained by using experimental data. In this study, it is taken as 0.7, which is a consistent value with the experimental data used.

$$V_{cr} = C_{cr} V_{\max} \tag{5}$$

Similarly, ultimate resistance (V_{ult}) is obtained as a fraction of V_{max} by using Equation 6. In this equation, parameter C_{sr} is introduced as the strength reduction factor, which is considered as 0.8 in this study. This means that the specimens that have lost 20% of their load resistance capacities are regarded as failed specimens.

$$V_{ult} = C_{sr} V_{\max} \tag{6}$$

Next, the displacement at cracking *(*δ_{cr}) is obtained by using Equations 7-8. In these equations, K_e is the effective stiffness of the wall that considers both flexural and shear deformations, H and I_w are wall height and moment of inertia, E_m and G_m are elasticity and shear modulus of masonry, respectively. Parameter β depends on the boundary conditions of the wall (3 for cantilever wall and 12 for fixed-ended wall) and parameter κ is the shear coefficient (taken as 1.2 for rectangular cross-section).

$$\delta_{cr} = \frac{V_{cr}}{K_e} \tag{7}$$

$$K_e = \left(\frac{H^3}{\beta E_m I_w} + \frac{\kappa H}{G_m A_w}\right)^{-1} \tag{8}$$

The other two displacement parameters δ_{max} and δ_{ult} are calculated by using secant stiffness (K_d) as a function of a damage index parameter (I_d) as shown in Equations 9-11 (Tomazevic 1999). In this equation, parameters c_1 and c_2 should be calibrated by using experimental data. In this study, these values are obtained as 1.28 and 0.32, respectively in accordance with the experimental data.

$$\delta_{\max} = \frac{V_{\max}}{K_d} \tag{9}$$

$$\delta_{ult} = \frac{V_{ult}}{K_d} \tag{10}$$

$$K_d = K_e - \sqrt{c_1 I_d - c_2} \tag{11}$$

For the calculation of δ_{max}, the damage index parameter, I_d is assumed to be 0.5, which corresponds to the attainment of maximum capacity of the wall with a network of diagonally oriented cracks in qualitative terms. In a similar manner, δ_{ult} is obtained by assuming I_d=0.75, which corresponds to extensive damage with widening cracks, crushing of the middle part of the wall or crushing and splitting at the compression and tension zones of the wall in qualitative terms.

Determination of limit state parameters for confined masonry walls is carried out in a slightly different manner since, unlike unreinforced masonry walls, several formulations exist in the literature for confined masonry wall parameters. For each limit state, formulations obtained for displacement and resistance parameters are compared with the available experimental data and the most suitable formulation is chosen to represent the corresponding idealized limit state parameter. The details of this study regarding the selection of suitable formulations for limit state parameters can be found elsewhere (Erkoseoglu 2014). In this paper, only the finally selected formulations to construct the idealized capacity curve for confined masonry walls are presented.

Just like in the case of unreinforced masonry walls, the procedure starts with the determination of V_{max}. For confined masonry walls, V_{max} is obtained using the formulation proposed by the masonry seismic code of Argentina (INPRES-CIRSOC 1983) since this is the formulation that provides the best prediction for the experimental values of maximum capacity. In this equation, parameter v_m denotes the basic shear strength of masonry.

$$V_{\max} = \left(0.6 v_m + 0.3 \sigma_y\right) A_w \tag{12}$$

For calculating V_{cr} *and* V_{ult}, the formulations used in the case of unreinforced masonry walls (i.e. Equations 5-6) seem to be the most suitable ones for confined masonry walls (Tomazevic 1999). In the case of confined masonry, parameters C_{cr} and C_{sr} are assumed as 0.7 and 0.8, respectively.

It is observed that the formulations proposed for displacement limit states of unreinforced masonry walls (i.e. Equations 7-11) also yield reasonable estimates for confined masonry walls. The only difference is that in Equation 11, parameters c_1 and c_2 are recalibrated for confined masonry wall data, and they have been obtained as 1.52 and 0.38, respectively.

For the verification of proposed formulations to construct idealized capacity curves of unreinforced and confined masonry walls, the ratios of calculated to experimental values for limit state parameters are obtained as presented in Tables 3 and 4. At the last row, mean values of the ratios are given. From Table 3, it is observed that all proposed formulations for displacement limit states of unreinforced masonry walls underestimate the experimentally obtained values on the average, 32% for cracking displacement, 14% for displacement at maximum resistance and 6% for ultimate displacement, respectively. On the other hand, the mean ratios for resistance limit states reveal that the predicted values slightly overestimate (6% for cracking limit, 2% for maximum response and 3%, respectively for ultimate limit) the resistance of unreinforced masonry walls. Overall, the results indicate that the seismic resistance of unreinforced masonry walls can be predicted better than the displacement capacity. This is not surprising, since it is easier to define and to test the resistance limits of rigid and brittle masonry components. That is why most of the seismic performance assessment methods for masonry structures use force-based approaches. It is a real challenge to obtain reliable measures to quantify the displacement capacity of unreinforced masonry components.

Final ratios of calculated to experimental values for limit state parameters for confined masonry walls are shown in Table 4. It should be noted that the match for displacement capacity of confined masonry walls is better due to the advantage of selecting among a number of different formulations whereas it gets worse in the case of resistance parameters. This may be due to the fact that the presence of vertical tie columns affects the failure modes and the overall behavior of masonry walls, so it is not possible to see pure failure mechanisms in the case of confined masonry walls. On the other hand, predicting the capacities for mixed failure mechanisms is a very difficult task and has not been considered in the equations used for this study.

Table 3. Ratios of calculated to experimental values for unreinforced masonry walls

Wall name	V_{cr}	V_{max}	V_{ult}	δ_{cr}	δ_{max}	δ_{ult}
URM-W1	0.94	1.01	1.03	0.94	1.17	1.23
URM-W2	1.19	0.95	1.18	1.27	1.06	1.06
URM-W3	1.11	0.95	1.08	0.97	1.23	1.06
URM-W4-1	1.46	1.23	1.48	1.38	0.99	1.21
URM-W5	1.40	1.33	1.21	0.51	0.86	0.98
URM-W6-1	1.26	1.26	1.15	0.48	0.65	0.73
URM-W7	1.04	1.04	0.90	0.66	1.17	1.78
URM-W4-2	1.55	1.19	1.02	1.07	1.27	1.67
URM-W8	1.13	1.07	1.07	0.46	0.77	0.98
URM-W6-2	1.13	1.13	1.13	0.39	0.63	0.76
URM-W9	0.33	0.38	0.38	0.76	2.20	1.43
URM-W10	0.54	0.77	0.77	0.88	0.58	0.40
URM-W11	0.35	0.42	0.42	1.08	0.45	0.52
URM-W12-1	1.07	1.07	1.07	0.42	0.40	0.44
URM-W12-2	1.07	1.07	1.07	0.42	0.46	0.53
URM-W13-1	1.11	1.11	1.11	0.37	0.61	0.82
URM-W13-2	1.05	1.05	1.05	0.37	0.77	0.80
URM-W14-1	1.16	1.16	1.16	0.35	0.58	0.72
URM-W14-2	1.11	1.11	1.11	0.44	0.58	0.70
URM-W15	1.16	1.16	1.16	0.42	0.70	1.03
Mean	1.06	1.02	1.03	0.68	0.86	0.94

Table 4. Ratios of calculated to experimental values for confined masonry walls

Wall name	V_{cr}	V_{max}	V_{ult}	δ_{cr}	δ_{max}	δ_{ult}
CM-W1	0.96	0.68	0.68	0.50	0.38	0.39
CM-W2	1.71	0.85	0.85	1.16	0.39	0.39
CM-W3	1.23	0.76	0.76	0.70	0.38	0.39
CM-W4	0.63	0.66	1.19	0.17	0.21	0.22
CM-W5-1	1.01	1.09	0.97	2.05	2.39	5.64
CM-W5-2	1.33	1.19	0.95	2.30	2.17	5.20
CM-W6	1.57	1.21	1.21	0.34	0.21	0.19
CM-W7	1.50	1.16	1.16	0.32	0.13	0.19
CM-W8-1	0.83	0.98	0.98	0.34	0.29	0.37
CM-W8-2	0.79	1.11	1.11	0.32	0.39	0.39
CM-W9-1	1.04	0.85	0.85	0.42	0.27	0.36
CM-W9-2	0.84	0.92	0.92	0.34	0.32	0.44
CM-W10	0.79	0.68	0.75	0.70	0.67	0.71
CM-W11	0.38	0.46	0.46	0.48	0.90	0.75
CM-W12	0.55	0.71	0.71	0.71	0.59	1.20
CM-W13	0.65	0.59	0.59	1.19	1.08	1.04
CM-W14-1	1.09	0.92	0.94	0.30	0.42	0.62
CM-W14-2	0.97	0.82	0.82	0.22	0.32	0.62
CM-W15	0.57	0.63	0.71	1.31	1.07	1.21
CM-W16	0.54	0.73	0.73	0.84	1.73	0.74
CM-W17	0.71	0.86	0.86	0.92	2.21	0.56
CM-W18	1.72	1.47	1.47	1.25	0.69	0.63
CM-W19	0.50	0.63	0.63	0.82	0.55	0.70
MEAN	0.95	0.87	0.88	0.77	0.77	1.00

The ratios in Tables 3 and 4 reveal that although there exist large discrepancies in values for some of the unreinforced and confined masonry wall tests, mean deviation from experimental results seems to be within acceptable limits. This shows that the formulations to predict limit state parameters of unreinforced and confined masonry walls can be used to idealize the behaviour of these walls.

PARAMETRIC STUDY REGARDING THE RESISTANCE OF UNREINFORCED AND CONFINED MASONRY WALLS

In the last phase, an analytical study is carried out to determine the effects of important parameters to resistance and deformation characteristics of unreinforced and confined masonry walls. Idealized models obtained in the previous section are used to calculate the force and deformation capacities of walls. For the parametric study, a set of curves for unreinforced and confined masonry walls are generated by using the aforementioned idealized capacity curve generation approach for variations of masonry compressive strength f_m =2 MPa (low strength), 5 MPa (moderate strength), 8 MPa (high strength); aspect ratio λ=0.5 (squat wall), 1.0 (square wall), 1.5 (slender wall); and axial compressive stress to masonry compressive strength ratio σ_0/f_m=0.05 (low stress), 0.10 (moderate stress) and 0.20 (high stress). In Figures, F and σ symbolizes f_m and σ_0/f_m, respectively. The hypothetical wall used in parametric study has 0.3 m thickness and 2 m length.

In Figure 3, the effect of masonry compressive strength (f_m) is assessed in square walls having equal length and height (λ=1.0) with different levels of axial compressive stress to masonry compressive strength ratio (σ_0/f_m=0.05, 0.10, 0.20). It is observed that for σ_0/f_m=0.05, behavior of unreinforced masonry walls is governed by flexure, but capacities due to diagonal tension resistance are very close. All the other walls are predicted to fail in diagonal tension. In unreinforced masonry walls, lateral load capacity V_{max} increases linearly with f_m. There is a similar trend in displacement capacity but the rate of increase slightly drops as f_m increases. This would indicate the more brittle nature of masonry specimens with higher strength. In case of confined masonry walls, similar effects are observed in lateral load and displacement capacities as f_m increases. The trends of the set of curves are more or less the same for λ=0.5 and λ=1.5.

If unreinforced masonry walls are compared to confined masonry walls, it can be observed that for σ_0/f_m=0.05, lateral load capacities are quite close to each other, but unreinforced masonry walls have a slightly larger capacity. As the axial load level is increased, the trend changes in the opposite way, i.e. the capacity of confined masonry walls begin to prevail. This reveals that under high axial load ratio, confined masonry walls have significantly greater capacity.

In all the cases, displacement capacities of confined masonry walls are larger than the capacities of unreinforced masonry walls for any level of f_m. This trend becomes more pronounced as the axial load level increases.

Next the effect of slenderness ratio (λ) on the capacities of unreinforced and confined masonry walls is assessed. The results are presented in Figure 4 for walls with moderate strength (f_m=5 MPa) and subjected to different levels of axial stress (σ_0/f_m=0.05, 0.10, 0.20). It is observed that unreinforced masonry walls with λ=1.0-1.5 and subjected to σ_0/f_m=0.05 are expected to fail in flexure, whereas all the other walls seem to fail in diagonal tension.

For unreinforced masonry walls, load capacity is very close in squat and square walls, but there is a significant reduction in capacity for slender walls. The same trend exists also for confined masonry walls, but the reduction in capacity is generally less. If unreinforced masonry and confined masonry walls are compared, lateral load capacities of confined masonry are generally higher, especially for the case λ=1.5 and σ_0/f_m=0.20. Results indicate that confined masonry construction improves the resistance of slender walls under high levels of axial stress.

Confined masonry walls have much greater deformation capacity than unreinforced masonry walls in every case, but the difference is slightly greater in case of slender walls and lowest in squat walls.

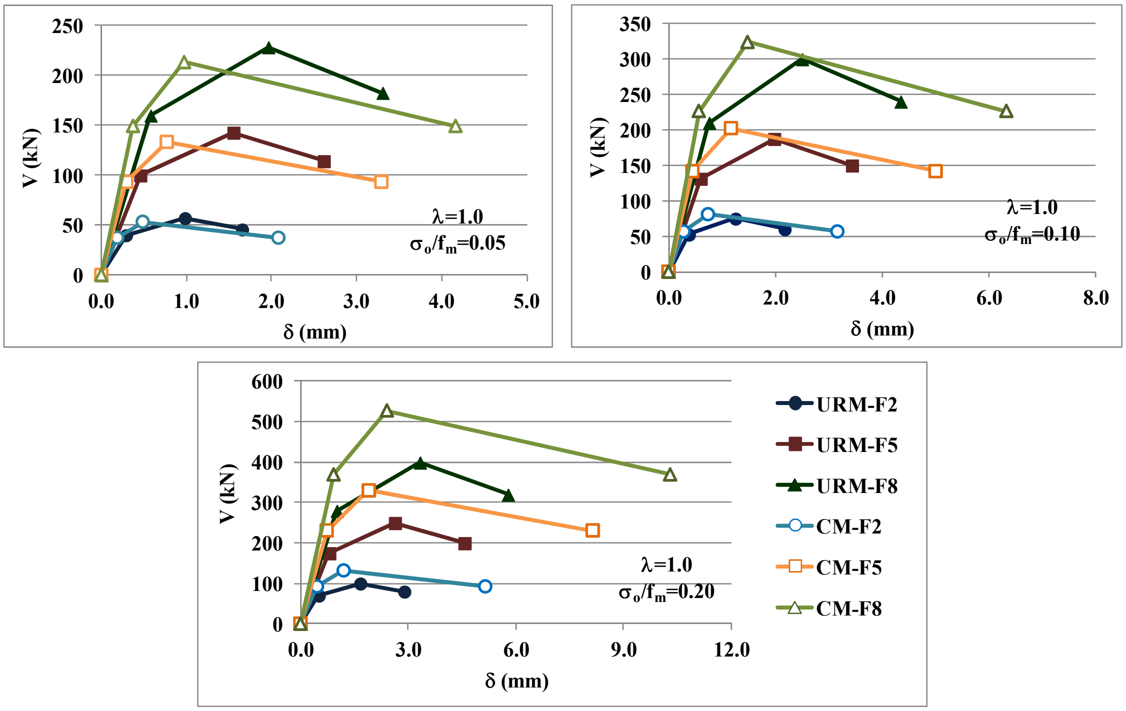

Figure 3. Effect of f_m on unreinforced and confined masonry walls for λ=1.0 and σ_0/f_m=0.05, 0.10, 0.20.

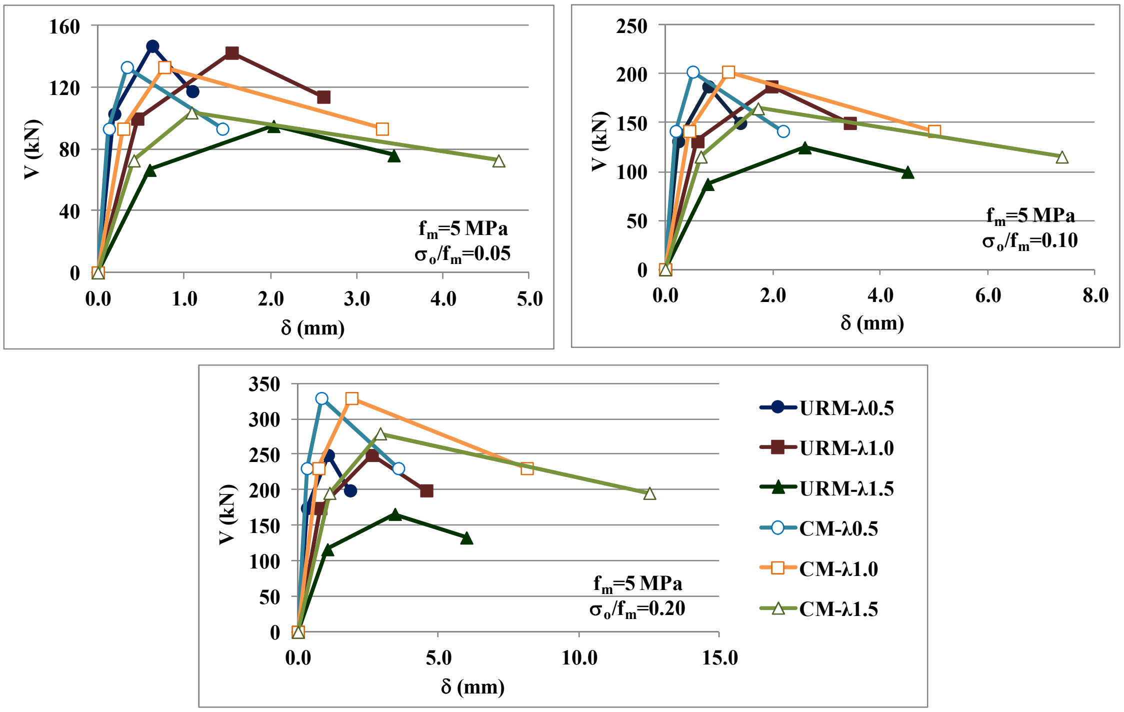

Figure 4. Effect of λ on unreinforced and confined masonry walls for f_m=5 MPa and σ_0/f_m=0.05, 0.10, 0.20.

Finally, the effect of axial stress to masonry compressive strength ratio (σ_0/f_m) on the capacities of unreinforced and confined masonry walls is assessed for the cases f_m=5 MPa and λ=1.0 as seen in Figure 5. It is observed that for unreinforced and confined masonry walls, lateral load capacity increases linearly with vertical stress ratio. But the increase in confined masonry walls is much more significant than unreinforced masonry walls. This trend seems to be more pronounced for slender walls. Deformation capacities show a similar trend as lateral load capacity. The increase in deformation capacity with σ_0 is almost linear and the rate of increase is much greater for confined masonry walls.

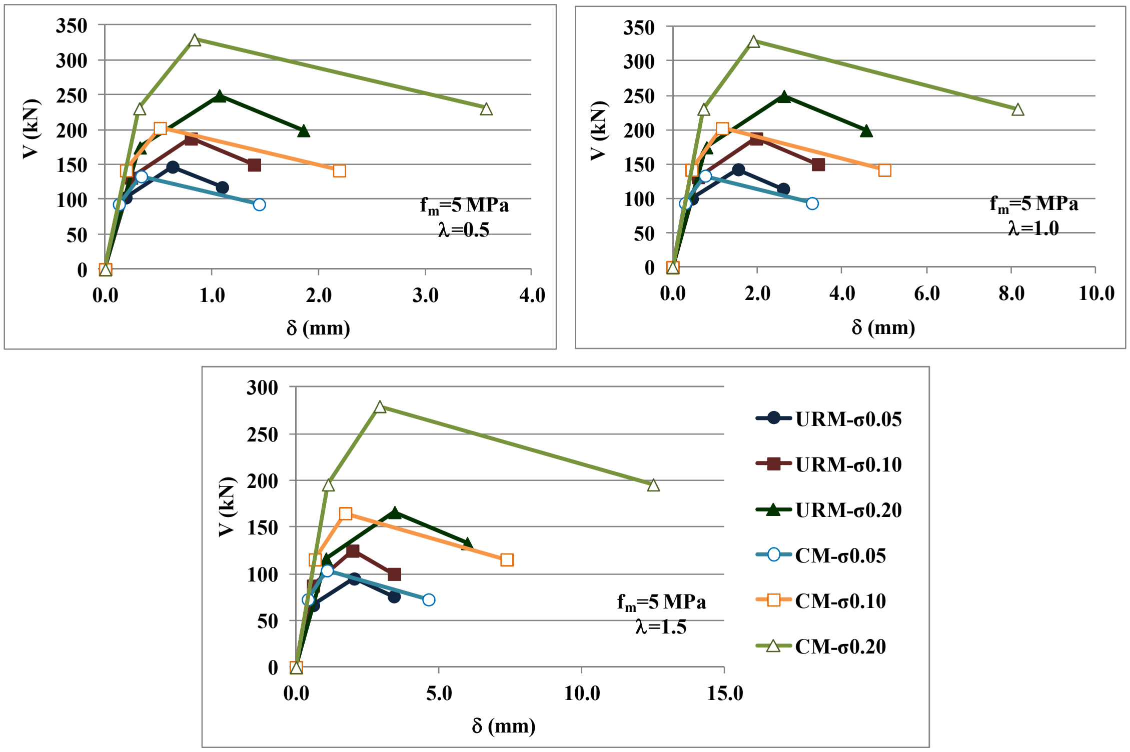

Figure 5. Effect of σ_0/f_m on unreinforced and confined masonry walls for f_m=5 MPa and λ=0.5, 1.0, 1.5

Overall, confining masonry walls induces a significant enhancement to lateral load capacity, which is in agreement with Kato et al. (1992), Yoshimura et al. (2004) and Riahi et al. (2009), but the enhancement is much greater in terms of deformation capacity, especially for walls subjected to high levels of axial stress. This observation is also in agreement with Gouveia and Lourenço (2007).

Looking at Figures 3-5 it can be stated that energy absorption capacity of confined masonry walls is always higher than that of unreinforced masonry walls. But there is a great variation between energy absorption capacity ratios of confined over unreinforced masonry walls. For a squat wall under low axial stress (i.e. representing a wall in upper stories of a building), energy absorption capacity ratio of confined over unreinforced masonry wall is close to 1.0 whereas for a slender wall under high axial stress (i.e. representing a wall in lower stories of a building) energy absorption capacity ratio of confined over unreinforced masonry wall exceeds 3.0.

CONCLUSIONS

The main goal of this study is to determine and compare the capacities of unreinforced and confined masonry walls by using an idealized trilinear analytical model. The limit state parameters of the model are determined by using the formulations proposed in the literature. In the case of having alternative formulations for a limit state parameter, the one with the closest match to the employed experimental database is selected.

In the final phase, a parametric study is carried out to predict the capacities of unreinforced and confined masonry walls. Masonry compressive strength (f_m), slenderness ratio (λ) and axial compressive stress to masonry compressive strength ratio (σ_0/f_m) are chosen to be the critical parameters. Effects of these parameters on the capacity of masonry walls are observed. and results are given in the previous section. According to the results, dominant failure mode of the walls seems to be diagonal tension failure. Even though square and slender walls under low levels of vertical stress are predicted to fail in flexure, the flexural capacities are very close to their capacities at diagonal tension failure. It is clearly observed that for high level of axial compressive stress, force and deformation capacities are significantly increased. This contribution is even higher as the wall gets slender.

Considering the limited number of test data, specific formulation, assumptions and simplifications used in this research and the results of the parametric study, the following conclusions are drawn:

- Confining columns increase lateral load capacity, ductility and energy absorption capacity of masonry walls. If the wall is not squat; under high levels of vertical stress, lateral load capacity is increased more than 30% and deformation capacity is increased more than 60% in confined masonry, which actually represent ground or lower story masonry walls in a masonry building. Since ground story is generally the critical one for the lateral load capacity of masonry buildings, this means the seismic resistance of a masonry building can be significantly enhanced by the introduction of confining tie-columns and beams. This can be regarded as an important feedback for the future seismic codes, especially for countries like Turkey, with the recommendation that only confined masonry construction should be used in highly seismic regions. This will bring no significant overburden to code-based design calculations, empirical design approach applicable for unreinforced masonry structures can still be valid with some modifications.
- When confined masonry walls are compared with the unconfined counterparts, the improvement in displacement capacity seems to be more pronounced than the improvement in strength capacity. In accordance with these observations, energy absorption capacity is also enhanced significantly. This means that the confining elements do not add much to the strength capacity of masonry walls, but they change the stress distribution and damage pattern in masonry walls in a positive manner. The increase in displacement and energy absorption capacity explains why confined masonry buildings experienced extensive damage but not collapsed in past major earthquakes while unreinforced masonry buildings were directly razed down.
- It seems to be a difficult task to idealize the seismic performance of masonry walls (for both unreinforced and confined) due to the complexity of the problem arising from the non-homogeneous, orthotropic and highly nonlinear nature of structural masonry. Most of the formulations in the literature to predict this behavior had been investigated in this study. These formulations have their own assumptions and simplifications, which may cause their deviation from the actual (experimental) behavior. It is much more challenging to find satisfactory results for all six limit state parameters considered in this study at the same time. There is not a unique set of formulations that can predict the seismic behavior of masonry walls with a very high accuracy for the whole range. That is why the predictions obtained for single walls are not good in some cases although the mean ratios are generally close to unity.
- The predictions concerning the strength capacity of masonry walls seem to be better than the predictions concerning the displacement capacity. This is not surprising since realization of displacement capacity of masonry walls is much more difficult due to many issues involved in

the process like unpredictable damage and crack patterns, mixed modes of behavior and failure, which cannot be quantified easily, calculation of inelastic displacements especially at the later stages of nonlinear behavior and localized damages like toe crushing or splitting. Hence it can be concluded that the idealized curves proposed in this study are much more suitable to be used in force-based design and analysis approaches, which is actually the current practice. However, the idealized curves should be used with caution in the case where displacement and performance based design and analysis approaches are to be considered.

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