A COMPARATIVE STUDY ON THE SEISMIC PERFORMANCES OF UNREINFORCED AND CONFINED MASONRY BUILDINGS

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ABSTRACT

Confined masonry (CM) buildings constitute a superior sub-class of masonry construction when compared to unreinforced masonry (URM) buildings in terms of seismic performance. Accordingly, this type of construction is very popular in South American Countries and in some regions of Asia. However until now, this has not been the case in Turkish construction practice, where nearly 30% of the total building stock is composed of URM buildings but a negligible percent of CM buildings. The main reason is to ignore CM construction in Turkey and not to promote it in the previous and the current seismic codes. But now, on the verge of releasing a new version of Turkish seismic code, there exist some attempts to adapt regulations concerning CM construction into the new code. This study tries to validate such attempts by comparing the seismic performances of URM and CM buildings starting from the component level up to the structural level. In order to assess the performances of these building types, seismic behavior of individual URM and CM walls had been estimated by using idealized models in the first phase of this research. In the second phase, this information is used to obtain seismic performances of URM and CM buildings. For this purpose, Capacity Spectrum Method is used. The results validate the superiority of CM construction over URM construction with a little effort by introducing a number of extra rules for CM detailing into the code.

Keywords: Unreinforced masonry; Confined masonry; Component level; Structural level; Capacity Spectrum Method

1. INTRODUCTION

According to the current construction practice, masonry construction has been overwhelmed by reinforced concrete (RC) construction in many regions of the world. Most people think that masonry is just an historical, rural or non-engineered construction type, which is highly vulnerable to seismic action. However, it should not be disregarded that properly designed and constructed masonry structures have some advantages over RC structures for low rise residential building construction such as low cost, durability, material availability, thermal isolation, fire resistance and low maintenance. Furthermore, they require little engineering technology and skill when compared to their RC counterparts.

Nowadays, there is an attempt to release the new version of the national seismic code in Turkey, also including the seismic design of new and assessment of existing masonry structures. In the current version (TEC 2007), CM construction is not explicitly included. There exist only empirical rules regarding URM construction. However, in most of the earthquake prone regions of the world including South America and Asia, CM construction is highly encouraged, even enforced. Hence, in order to promote this specific type of masonry construction on the verge of releasing a new seismic code in Turkey, this study focuses on comparing the seismic performances of these two sub-classes of masonry construction (i.e. URM and CM) at both component and structure levels and to quantify the

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superiority of CM construction over traditional URM construction through a parametric study. Although it may seem to be obvious that CM construction is superior to the URM counterpart, the comparative results obtained in this study are still valuable since they quantify the degree of superiority of this type of construction and the role of different structural parameters in this quantification.

2. URM AND CM BUILDING MODELS

Prior to the development of the building models (in the structural level) for both URM and CM constructions, the wall models (in component level) for these construction types should be established in the first phase of the study. The capacity curves of these wall models were obtained in a previous study (Erkoseoglu 2014) by employing idealized piece-wise linear capacity functions. Capacity curve parameters for this idealization were selected from the analytical and empirical equations given in the literature and calibrated by the existing experimental data. Then a parametric study was conducted by using the idealized capacity curves of these two masonry wall types in order to compare their seismic performance for variations in masonry compressive strength $f_m$=2 MPa (low strength), 5 MPa (moderate strength), 8 MPa (high strength); aspect ratio $\lambda$=0.5 (squat wall), 1.0 (square wall), 1.5 (slender wall); and axial compressive stress to masonry compressive strength ratio $\sigma_0/f_m$=0.05 (low stress), 0.10 (moderate stress) and 0.20 (high stress). The hypothetical wall component used in parametric study has 0.3 m thickness and 2 m length.

Figure 1 represents the effect of compressive strength ($f_m$) on URM and CM walls for $\lambda$=1.0 and $\sigma_0/f_m$=0.05, 0.10, 0.20. In the legends of these figures, F symbolizes $f_m$. It is observed that increase in maximum strength is proportional to $f_m$ for both URM and CM walls. Ultimate displacement values show a similar trend. When URM and CM walls are compared, CM walls seem to have higher strength capacity than URM counterparts for all levels of axial stress. Failure of URM walls seems to occur by flexure under low axial stress but in other cases diagonal tension governs the failure mode. CM walls are all assumed to fail in diagonal tension.

![Figure 1](image-url)

Figure 1. Idealized capacity curves for URM and CM square ($\lambda$=1.0) walls with varying compressive strength values ($f_m$=2, 5, 8 MPa) subjected to vertical stress ratios $\sigma_0/f_m$=0.05, 0.10, 0.20.
In this paper, only the effect of $f_m$ on URM and CM walls has been presented due to page limitations. The details and the complete discussions regarding the parametric study can be found elsewhere (Erkoseoglu 2014). However, it can be summarized that in all cases discussed in this reference study, CM walls showed better performance when compared to their URM counterparts under all different parametric combinations. Briefly, this verifies the superiority of CM construction in the component level over URM construction.

In the second phase of the study, generic URM and CM buildings models are developed by using the idealized wall models obtained in the previous phase. For URM models, the geometry and distribution of walls within the story plan, position of openings and the amount of walls in both orthogonal directions are determined by considering the principles of the existing Turkish earthquake code (TEC 2007). For CM models, especially for the placement of vertical ties within the story plan, seismic design principles of building codes from several countries have been taken into account (Argentinean Seismic Code 1983, INN 1997, MHUV 2003, Mexican Building Code 2004). In all of these countries, design and construction of CM buildings are common practice and explicitly encouraged by the relevant codes.

For the building models, 6 generic story plans for each construction type (in total 12 story plans) are developed. The model names are coded in two parts. The first letter stands for the type of construction, i.e. U for URM models and C for CM models, followed by a hyphen. After the hyphen, the next four parameters are introduced in an alpha-numerical way, i.e. each with a letter and a number. The first parameter is number of stories, abbreviated with “N”. In this study, 2 and 3 story building models have been studied, so the coding for number of stories becomes N2 or N3. Next parameter is related to the wall distribution of the building in plan, classifying models as having “regular” or “irregular” wall distribution. This is abbreviated with “R”. Regular wall distribution (i.e. R1) means having the masonry walls distributed evenly in plan. Irregular wall distribution (i.e. R2) means having the masonry walls distributed unevenly in plan, which may create significant torsional effects within the story. The third parameter is the compressive strength of masonry walls ($f_m$), which is abbreviated by “F”. In this study, three levels of strength have been considered in accordance with the values taken for the parametric study of walls (i.e. $f_m$=2, 5, 8 MPa). These three levels are classified as “F2”, “F5” and “F8” for the building models. The final parameter is related to the required length of walls in any principal direction and size and distribution of openings in walls in the critical story. This parameter is abbreviated by “W” and it is considered in three different classes as “W1”, “W2” and “W3”. The definitions of these sub-classes are based on the criteria given in TEC (2007). The major criterion is the $L_d/A_g$ ratio, which is defined as the ratio of the total length of masonry load bearing walls in any of the orthogonal directions in plan ($L_d$) to the gross area ($A_g$). A certain limit is enforced in the code for this criterion, which is 0.2 for residential buildings in TEC (2007). Other criteria are minor in the sense that they are related to required wall lengths between openings or between openings and corners, etc. in a local sense. Accordingly, W1 stands for the story plans which do not violate the code principles and possess adequate lateral wall resistance whereas W3 represents the story plans which do not obey most of the code principles and possess inadequate lateral wall resistance.

The story plans, of which two are demonstrated in Figure 2, are for all the variants of R and W classes (i.e. R [2 classes] × W [3 classes] = 6 variants for each construction type). The other parameters (N and F) are not directly related with the story plans. In total, there exist 2 (U or C) × 2 (N2 or N3) × 2 (R1 or R2) × 3 (F2, F5 or F8) × 3 (W1, W2 or W3)=72 building models to be analyzed. The building models are coded as (U or C)-N(2 or 3)R(1 or 2)F(2, 5 or 8)W(1, 2 or 3). For instance, a 2-story regular URM building model with $f_m$=5 MPa and conforming to wall length and distribution criteria given in the code is abbreviated as U-N2R1F5W1.

It should be noted that models with wall irregularity do not conform to the standards. For all the models, concrete slabs and tie beams are assumed to be present for rigid diaphragm action. Tie beams have a width equal to wall thickness and height of 15 cm according to Eurocode 6 (2003). Floor height is taken as 2.80 m, which satisfies 3 m criterion in TEC (2007). For CM models, tie columns are placed principally at the corners of all the openings, but for some small wall segments that remain on...
the corners and intersections of walls, they are not used because those small wall segments do not make a significant contribution to the lateral load capacity and not considered as load bearing walls. Hence these walls are not taken into account in the lateral load capacity evaluation of the critical storey.

Figure 2. The floor plans of building models U-R1W2 and C-R1W2 (all dimensions in cm)
3. THE EMPLOYED PERFORMANCE ASSESSMENT METHODOLOGY

In order to predict the seismic behavior of generic URM and CM building models in this study, improved Capacity Spectrum Method (Procedure C) proposed by FEMA 440 (ATC 2005) is used. The procedure is based on finding the performance point by plotting the demand curve (i.e. earthquake design or response spectrum) and capacity curve (i.e. storey shear capacity) on the same graph in ADRS (Acceleration Displacement Response Spectra) format. This format requires using spectral displacement ($S_d$) and spectral acceleration ($S_a$) values. To convert the capacity curve into ADRS format, base shear ($V$) should be converted into $S_a$ and roof displacement ($\Delta_{\text{roof}}$) should be converted into $S_d$.

For quantification of seismic demand, 10 ground motion records from different earthquakes with magnitudes ranging between 5.8 and 7.8 have been used in this study. Elastic acceleration spectra of the selected ground motion records are provided in Figure 3 (gray lines). The mean and mean±one standard deviation curves (black lines) are also presented in the same figure. The spectral curves seem to exhibit a large scatter in the short period region intentionally to emphasize the record-to-record variability with variable ground motion characteristics. In the selection of these records, the main criterion has been peak ground acceleration (PGA) since it is a known fact that rigid masonry structures are highly influenced from this ground motion parameter. The records have been selected in such a way that their PGA values cover all levels of ground motion intensity, from 0.06g to 0.75g.

![Figure 3. Elastic acceleration spectra of the selected ground motion records together with their mean and mean±one standard deviation curves](image)

For quantification of seismic capacity, pushover curve of the critical storey is constructed by using the idealized capacity curves of individual wall segments as explained in the previous section. This is achieved by superimposing the capacity curves of individual wall segments in any principal direction of the critical storey to obtain the overall envelope. There exist three gross assumptions while applying this approach. First, the floors are assumed to be rigid in their own planes so that the lateral forces are distributed to the wall segments in proportion to their relative stiffness. This is generally valid in the case of RC floors, but questionable in the case of wooden floors for masonry buildings. In this study, the floors are assumed to be rigid RC floors. The second assumption is that the first mode shape dominates, or in other words, lateral floor displacements can be calculated by using this mode shape. A further simplification for the method is to use inverse triangular distribution instead of the first mode shape since they can be assumed to be very close to each other. In this way, it becomes simple to relate ground storey displacement to the roof displacement. The third assumption is that the flange effect of orthogonal walls connecting to a load bearing wall is neglected.

A displacement based procedure is applied to the building models under consideration to construct the pushover curve. The process is carried out by gradually pushing the building model, namely; increasing the displacement by a small increment in each iteration and calculating the storey shear capacity. Displacement in the direction of analysis is augmented in small increments. In each step,
shear capacities of considered wall segments are summed up to yield the storey shear capacity. Torsional moments due to the eccentricity between mass and rigidity centers are also calculated. Accordingly, the change in deformation due to the rotation is calculated for each wall and capacities of wall segments are recalculated according to their final displacement values. By using final displacement values, total shear capacity of the critical storey is obtained. Displacement is increased up to a point that one or more wall segments reach their ultimate displacements, i.e. the point corresponding to 80% of maximum strength. At that point, pushover procedure restarts with zero displacement, but the wall segments that have reached to their ultimate displacement in the previous pushover analysis are not taken into account. Multiple capacity curves are plotted and they are combined such that each curve starts at the point where the previous curve ends. At the end, a saw-tooth shaped storey shear capacity curve is obtained as seen in Figure 4 in accordance with FEMA 440 (ATC, 2005).

![Figure 4. A typical saw-tooth shaped storey shear capacity curve obtained in this study](image)

After the demand and capacity curves are converted into ADRS format, the last step is to find the performance point by applying Procedure C in FEMA440 (Figure 5). The procedure is based on adjusting initial earthquake spectrum in ADRS format for different levels of ductility (µ). To make this adjustment, first effective damping (βeff) is calculated for different ranges of ductility. The details of the proposed formulations can be found in Chapter 6 of FEMA440.

In the final stage of the Capacity Spectrum Method, superposition of seismic demand with capacity yields the performance points of the 72 building models subjected to 10 ground motion records of different seismic intensity levels, summing up to 720 cases of performance analysis for URM and CM building models.

![Figure 5. Locus of possible performance points using the considered FEMA procedure (ATC, 2005)](image)
Once the performance points on the storey shear capacity curves are determined, performances of all load carrying wall segments in the critical storey level can be obtained. These performances are assessed according to the aforementioned idealized trilinear capacity curves. For each wall segment, 3 different limit states (LS), and therefore, 4 different performance states (PS) are defined as shown in Figure 6. The first performance state (PS₁ in Figure 6) represents the behavior of the masonry wall segment before cracking initiates. In this state, the masonry wall behaves in the elastic range. The second performance state (PS₂ in Figure 6) represents the behavior of the masonry wall segment that has reached the cracking load, but not the maximum shear resistance. In this state, the masonry wall segment starts to show inelastic behavior but the crack width and propagation is not significant, therefore the damage level is low. The third performance state (PS₃ in Figure 6) represents the behavior of the masonry wall segment that has reached its maximum resistance but it has not been pushed to its ultimate deformation capacity yet. Although the crack distribution and damage level is significant, the wall segment is still able to maintain more than 80% of its maximum resistance. Finally, the fourth performance state (PS₄ in Figure 6) represents the failure state. Increasing deformations cause the wall segment to lose its integrity. The resistance of the wall segment drops to less than 80% of the maximum resistance. In this state, wall segment is no longer a lateral load carrying member and the loads carried by that member should be distributed to other members.

In order to evaluate the overall performance of the critical story in the building, the information regarding the existing states of all the walls in that story should be gathered. The most appropriate way to determine the overall performance state of the critical storey of a building is to employ a damage index (DI) with weighing factors. For this purpose, the weighing factors (wᵢ) are assigned to each performance state as indicated in Table 1. Hence the damage index (DI) can be defined as seen in Equation 1.

\[
DI = \frac{\sum_{i=1}^{n} w_i k_i}{\sum_{i=1}^{n} k_i} \quad (1)
\]

In this equation, \( n \) is the number of wall segments in the direction of analysis; and \( k_i \) is the in-plane stiffness of the wall segment \( i \). The significance of using the in-plane stiffness of the walls in Equation 1 is based on the assumption that the story shear is distributed with respect to the relative in-plane stiffness of the walls in that story. Hence as the wall gets stiffer, it attracts larger forces. This increases the importance of that individual wall within that story, which should be reflected in the damage index in an explicit manner. The damage index takes values between 0 and 1, for which the lower bound means that all the walls in the critical story behave in their elastic range under the specified seismic action whereas the upper bound means that all the walls in the critical story have failed during the
seismic action. Generally, depending on the performance states of the walls, DI takes values in between the bounds.

The procedure explained in this section is used to compare the relative performances of generic URM and CM buildings subjected to different levels of ground motion records in the next section.

Table 1. Weighing factors defined for the performance states

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<tr>
<th>Performance state</th>
<th>Weighing factors</th>
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<td>PS₁</td>
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<td>PS₄</td>
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4. EVALUATION OF ANALYSIS RESULTS

In this section, the analysis results obtained from the Capacity Spectrum Method are evaluated for the comparison of seismic performances for URM and CM models as follows:

- When compared with each other, the DI values clearly show the superiority of CM models over URM models for almost all cases. This is clearly observed in Figures 7.a and 7.b for 2-story and 3-story URM and CM building models with the most favorable (regular plan geometry, high strength, adequate masonry walls with even distribution, i.e. R1F8W1) and unfavorable (irregular plan geometry, low strength, inadequate masonry walls with uneven distribution, i.e. R2F2W3). Even in the case of building models with totally opposite parameters, CM variants seem to be performing better than URM variants with lower DI values and higher PGA values at DI=1.0.

- It is also possible to comment on the effect of number of stories (N2-N3) from Figures 7.a and 7.b. It can be stated that DI values are sensitive to number of stories in the case of URM models, especially for most favorable parameters (i.e. R1-F8-W1). In the case of most unfavorable parameters (i.e. R2-F2-W3), N-curves get closer to each other. In the case of CM models, DI is slightly sensitive to number of stories for R1-F8-W1 whereas it does not seem to be sensitive for R2-F2-W3 as the N-curves overlap for nearly all levels of PGA.

- The influence of material strength (i.e. F2-5-8 cases) on the seismic performance of masonry buildings is examined in Figures 8.a and 8.b for regular and irregular 3-story models with adequate and inadequate length and distribution of walls in the critical story (i.e. N3-R1-W1 and N3-R2-W3, respectively) as the most unfavorable and favorable parameters for this

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![Figure 7](image_url)  
**Figure 7.** DI-PGA relationships for building models with a) 2 and b) 3 stories
comparison. For both cases, URM models with low strength reach to collapse state (DI=1.0) at low levels of PGA (i.e. around 0.2g), followed by models with moderate strength reaching DI=1.0 at approximately 0.4g. The URM models with high strength take the value of DI=1.0 when PGA~0.6g. This trend shows the importance of strength parameter on the seismic performance of URM models. CM models never reach to the value of DI=1.0. This trend shows that CM models behave better than URM models under similar conditions. Furthermore, the strength parameter is relatively less pronounced for CM models. This is due to the fact that there are different parameters in CM construction that play important roles for seismic performance other than the material strength.

Figure 8. Sample DI-PGA relationships for the comparison of F-parameter

- The effect of other two parameters (wall distribution in plan or R-parameter, total length of walls and their distribution within the story plan or W-parameter) on seismic performance of URM and CM models have also been considered. There seems to be slight influence of these parameters for URM models and no influence for CM models. The details of these analysis can be found elsewhere (Citiloglu 2016).

Another important parameter to assess the seismic performance of URM and CM models is the PGA value at collapse (DI=1.0), i.e. PGA_C. This parameter is defined as the acceleration value when DI=1.0 for the first time. Examining the PGA_C values for URM and CM models, it is observed that most of the URM models (i.e. 83%) reach at the collapse state at some PGA value whereas only 1 out of 36 CM models (i.e. 3%) reaches at the collapse state. This shows the overwhelming superiority of CM models over URM models in terms of reaching the collapse state during seismic action. CM buildings are more immune to collapse due to their modular structure, where the vertical tie-columns, horizontal tie-beams and the wall segments enclosed by these RC non-structural elements constitute the modules of the building.

In order to evaluate the PGA values at collapse for the URM models, they are classified into three sub-classes according to the quality of construction as “good”, “moderate” and “poor”. URM models with good quality of construction represent regular or nearly regular buildings with good material quality and strength, having adequate amount of and evenly distributed masonry load-bearing walls in plan. Such buildings are generally located in urban regions, made of massive stone, good quality brick or new technology materials like autoclave aerated concrete (AAC). In this study, such structures are represented by the following eight URM building models. Figure 9 illustrates PGA_C values of these eight models. The average PGA_C value of the remaining six building models is 0.54g±0.10g (i.e. COV=0.19).
URM models with moderate quality of construction represent regular or irregular buildings with variable (but on the average moderate) material quality and strength. These buildings may have deficiencies in relation with the amount or story plan distribution of masonry load-bearing walls. Such buildings constitute the majority of the URM building stock, either in urban or rural regions. In this study, such structures are represented by the following twenty URM building models. Figure 10 illustrates PGA<sub>C</sub> values of these twenty models. The average PGA<sub>C</sub> value of the remaining sixteen building models is 0.42±0.15g (i.e. COV=0.34).

Finally, URM models with poor quality of construction represent irregular buildings with low material quality and strength. These buildings have major deficiencies in relation with the amount or story plan distribution of masonry load-bearing walls. Such buildings are generally located in rural regions or in suburb regions of cities. This group of buildings is known to exhibit low seismic performance even in moderate size earthquakes. In this study, such structures are represented by the following eight URM building models. Figure 11 illustrates PGA<sub>C</sub> values of these eight models. The average PGA<sub>C</sub> value of these eight building models is 0.30±0.10g (i.e. COV=0.32).
Figure 11. PGA\(_C\) values of URM models classified as “poor quality construction”

The above information is valuable in the sense that it gives a crude estimation of the range of PGA values that would cause the collapse of the URM building class under consideration. As expected, poor quality and deficient URM structures are the most vulnerable ones with a PGA value range 0.20g to 0.40g to induce collapse whereas well-constructed and code-compliant URM buildings have much higher PGA values at collapse state (0.40g to 0.62g). The moderate quality URM buildings that constitute the majority of the building stock have a PGA range of 0.29g to 0.57g, depending on the structural properties. The high COV value of this class of URM buildings arise from the wide range of properties inherent in this group. Hence it can be concluded that deficient URM structures exhibit heavy damage or collapse even under moderate levels of ground motion intensity (i.e. PGA=0.2g). As the quality of construction increases and structural deficiencies decrease, URM structures perform better and PGA values at collapse rise up to values of 0.5g-0.6g. On the other hand, CM buildings have much more better behavior than URM structures and exhibit almost no collapse under the levels of seismic action considered in this study.

5. CONCLUSIONS

Based on these assumptions and simplifications considered in this study, the following conclusions are drawn:

- Low-rise CM buildings have very good seismic performance even under strong seismic action. They are more immune to collapse due to their modular structure, where the vertical tie-columns, horizontal tie-beams and the wall segment enclosed by these RC non-structural elements constitute the modules of the building. Each module behaves in an independent manner since the damage and crack formation cannot propagate from one wall to the other (as in the case of URM buildings) due to confining elements in between. This increases the energy dissipation capacity of the CM buildings, which in turn decreases the possibility of experiencing collapse during seismic action. Although some of the wall segments (or modules) confined by tie-beams and tie-columns can be heavily damaged during shaking, this may not induce total collapse since the structure is still stable due to non-damaged or slightly damaged wall segments.
- On the other hand, URM buildings seem to be vulnerable to seismic action even under moderate levels of seismicity, especially in the presence of unfavorable structural parameters like low strength, inadequate amount and uneven distribution of masonry walls, etc.
- The structural parameters considered in this study have some influence on seismic performance in the case of URM buildings. Especially, masonry strength has a major effect.
However, in the case of CM buildings, they do not seem to have a considerable effect on the seismic performance. This may be due to the fact that there are different parameters in CM construction that play important roles for seismic performance other than the ones considered in this study.

- Due to its good seismic performance observed in this study, CM construction should be encouraged in Turkey, especially for small-to-medium sized low-rise residential buildings. The only way to achieve this is to promote this type of construction in the new version of the Turkish Earthquake Code. For instance, the new code may enforce the construction of only CM buildings in regions of high-seismicity whereas URM buildings are allowed to be constructed in regions of low-to-moderate seismicity. In this way, it may be quite possible to save lives of many in rural and suburb regions only by obeying some simple construction principles regarding CM buildings. This issue seems to have been put into action in most of the earthquake-prone South American countries by enforcing the construction of CM buildings through legislation and shifting the content of the building stock from more vulnerable URM buildings to CM buildings. Besides, according to a study by Marques and Lourenço (2014), for a typical two storey house, cost of CM structure is 16% less than the RC structure. This is another reason why low-rise CM construction should be encouraged.

- Simple methods and approaches are very appropriate for seismic analysis of masonry structures when compared to the classical time-history analysis. In this way, it becomes possible to conduct a large number of analyses (which is not possible to do with the classical methods) and carry out parametric studies on masonry behavior. However, since such methods and approaches possess many assumptions and simplifications (as also present in this study), the results obtained in this study should be supported by laboratory tests and the actual performance of URM and CM buildings in the field.

6. REFERENCES


