Performance assessment of low–rise confined masonry structures for Earthquake induced ground motions

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ABSTRACT

The performance of low-rise confined masonry (unreinforced masonry walls confined with horizontal and vertical lightly reinforced concrete elements) structures is assessed against earthquake induced site amplified strong ground motions using a probabilistic-based approach. This included analytical investigation of field representative structural models through a suite of natural accelerograms, for various hazard levels. The basic mechanical characteristics of structural material is obtained through experimental investigations recently carried out on masonry material, structural walls and reduced scale structural models, which are employed for the design, mathematical modeling and seismic analysis of confine masonry structures. The seismic performance of two case study (two storey) structure types is assessed for various scenario earthquakes with moderate to strong ground motions. The structures are designed and analyzed considering the existing building stock and new construction recommendations; both differ in provisioning of basic mechanical properties of building material for construction. The typical confinement of masonry walls can avoid the total structural collapse in most of the strong ground motions thereby minimizing the occupant’s injuries, however the damages to structures in large earthquake events are significant. Besides the good behavior offered by confinement scheme, the role of construction material’s mechanical properties in performance improvement is significant.

Keywords: Confined Masonry Structures; Earthquake-Resistance; Seismic Performance; Construction Deficiencies.

1. Introduction

1.1 Motivation of the Study

Most of the developing and underdeveloped parts of the world primarily make use of traditional masonry construction for residential purposes due to the fact that people in these countries are drawn towards making self-build dwellings of traditional practice and materials. The traditional construction practice is also motivated by the climatic condition of the country where the people need massive roofs and walls to protect the inside of dwellings from the extremity of outside temperature and high humidity. Significant differences may exist also in day time and night time temperature in a given season. Lessons from past and recent earthquakes has demonstrated that, traditional structure types can provide significant resilience to earthquake induced strong ground shaking, if constructed in a good engineering sense (Ahmad et al., 2011; Ali, 2007; Langenbach, 2010; Magenes, 2006). In other cases masonry structures may collapse in a catastrophic manner which can result in the loss of life and economy (Naseer et al., 2010; Rossetto and Peiris, 2009; ADB, 2005). However, in most
of the past and recent earthquake events in Latin America the performance of confined masonry structures (masonry structures in which walls are confined using vertical and horizontal lightly reinforced concrete element) was found appreciable, even when constructed with modest effort, when subjected to ground shaking in the absence of secondary effects like liquefaction and landsliding during earthquake (Brzev, 2007; Dowling, 2004; Lang and Marshall, 2011; Moroni et al., 2004; Schultz, 1994 among others). Very few cases of poor performance of confined masonry structures has been noticed, when gross construction errors, design flaws, or material deficiencies have been introduced in the building design and/or construction process (Brzev, 2007), which however needed to be addressed in the design and assessment of confined masonry structures for their likely impact on the global behavior of structures.

1.2 Confined Masonry Construction

Confined masonry construction was probably first introduced in the reconstruction activities following the devastating Mw 7.2 1908 Messina earthquake in Italy. However, it can be found now in various parts of the world with high seismic hazard e.g. Europe (Italy, Slovenia, Serbia), Latin America (Mexico, Chile, Peru, Colombia, Argentina, and other countries), the Middle East (Iran, Algeria, Morocco), South Asia (Indonesia), and the Far East (China) (Meli et al., 2011). Observing the good behavior of confined masonry construction during various past earthquakes the technique is now recommended and adopted by many local national authorities, after being struck by a devastating earthquake, for reconstruction activities, rapid recovery of the community and a mean for sustainable and earthquake resilient construction (CMN, 2010). Similarly, following the recent Mw 7.6 2005 Kashmir earthquake, the local government has primarily recommended the confined masonry construction technique for building residential houses in major affected areas of Pakistan and prepared easy to follow guidelines for facilitating construction projects which is adopted by many local builders and home owners for reconstruction of buildings destroyed in earthquake (Stephenson, 2008).

Figure 1 shows typical confine masonry construction schemes practiced in Pakistan after ERRA (2007), Schacher (2007) and Shahzada (2006). However, like most of other countries the guidelines are prepared and disseminated without knowing much about the likely performance of a given structural system in future expected earthquakes and the negative impact of deficient construction if any, which is essential.

1.3 Scope and Objective of the Study

This paper presents investigation on typical low-rise confined masonry construction structural schemes practiced in the major affected areas of Pakistan in order to understand their likely behavior during strong ground motions, in a probabilistic fashion, from engineering stand point. Besides the construction scheme, the impact of construction deficiencies like the use of low quality building material i.e. masonry with lower mechanical properties, on the seismic performance of structures is also assessed. This is particularly important for existing building stock in Pakistan and new construction in case low quality material is used in the building construction process. Experimental investigations carried out recently at the Earthquake Engineering Center and University of Engineering and Technology (UET) Peshawar on confined masonry material and reduced scale structural models (Naseer, 2009; Shahzada et al., 2011) are considered for the design and mathematical modelling of structures investigated herein.
2 Seismic assessment of Confined Masonry Structures

2.1 Characteristics of Structures Investigated in The Present Study

For case study investigation two structure types with simple geometric detailing are obtained from Ahmad et al. (2012), as shown in Figure 2, representing typical plan of structures currently practiced in major earthquake affected regions. In the first phase, the structures were first designed considering the basic material properties of existing building stock, labeled as CM1\textsubscript{Old} (left) and CM2\textsubscript{Old} (right), which are consequently designed with the material properties recommended for new construction, labeled as CM1\textsubscript{New} (left) and CM2\textsubscript{New} (right). CM1 structure type is considered with floor area of 50 square meter and wall density of 4.20 percent in the excitation direction whereas CM2 is considered with floor area of 75 square meter and wall density of 4.60 percent.

The ground floor height of structures is 3.50m whereas the first storey has interstorey height of 3.00m. The structures are provided with deep spandrels of depth 1.50m on each floor. The height of windows and doors opening is considered as 1.00m and 2.00m respectively. For CM1 structures the length of corners short piers is 1.00m, the middle piers have length of 1.50m and the intermediate piers are 0.90m in length. For CM2 structures the length of corners short piers is 1.30m, the middle piers have length of 1.90m and the intermediate piers are 1.15m in length. Both the structures are considered with confining columns provided at the openings, corners and wall junctions with typical cross-section size of columns: 152mmx229mm at the end of wall and openings, 229mmx229mm at the wall junctions. The structures are provided with bond beam with typical cross-section size of 152mmx229mm. Each of the confining elements are reinforced with 4ф12mm longitudinal bars which are...
provided with transverse reinforcement (stirrups) of \(1\Phi 10\) mm at 150mm centre-to-centre distance. The basic material properties considered for each of the structure type and constituent are reported in Table 1.

![Figure 2: Typical geometric detailing of example structures investigated in the present research study. From left to right: CM1 and CM2](image)

**Table 1:** Basic material properties of confined masonry structures investigated in the present research study, after Shahzada et al. (2011) for existing building stock and Naseer (2009) for new recommendation

| Property                              | Observation                        |
|---------------------------------------|------------------------------------|
| **Old (Existing Stock)**              | **New (Recommendations)**          |
| Masonry Unit Dimensions               | 108mm x 221mm x 67.5mm             | 107mm x 222mm x 70mm              |
| Unit Compressive Strength             | 12.43 MPa                          | 16.10 MPa                         |
| Mortar Compressive Strength           | 5.05 MPa                           | 6.90 MPa                          |
| Masonry Compressive Strength          | 2.61 MPa                           | 5.80 MPa                          |
| Masonry Diagonal Tensile Strength     | 0.05 MPa                           | 0.35 Mpa                          |
| Concrete Compressive Strength         | 12.07 MPa                          | 10.030 MPa                        |
| Yield Strength of Reinforcing Bars    | 327.58 MPa                          | 652 MPa                           |
| Masonry Modulus of Elasticity         | 1228 Mpa                           | 1980 Mpa                          |

**2.2 Seismic Analysis Framework**

A fully probabilistic and nonlinear dynamic reliability-based method (NDRM) is employed for the seismic performance evaluation of structures. The approach has been tested and validated against real earthquake observations, which provided reasonable estimate on the structures performance during moderate and large magnitude earthquakes (Ahmad, 2011). The approach included a nonlinear dynamic reliability based methodology for calculating the damage exceedance probability of a specified ground motions i.e. estimating the chances a specified ground motion that can cause a particular damage in the structure. In the first step,
the structures are analyzed for a selected suite of natural accelerograms using incremental dynamic analysis (IDA) technique (Vamvatsikos and Cornell, 2002) in order to derive structure response curve by correlating the shaking intensity with the structural demand (drift demand) with due consideration of the likely uncertainties. The drift demand on structure is convolved with the structure capacity (drift capacity) to obtain the exceedance probability of specified damage state at specified shaking level, see Figure 3.

**Figure 3:** Probabilistic framework for seismic assessment of structures, after Ahmad (2011). From left to right and top to bottom: mathematical modelling of structure, selection and scaling of accelerograms, derivation of seismic response curves for structures and estimation of exceedance probability of ground motions for a specified performance level and shaking intensity.

3. Mathematical Modeling of Confined Masonry Structures

3.1 Response Mechanisms (Failure Modes) of Confined Masonry Walls

A masonry structure subjected to an earthquake are excited in both principal directions whereby structural wall components are subjected to lateral in-plane (right through the wall length) and out-of-plane (parallel to wall length) loading. The predominant mode of failure governs the seismic capacity of a structure i.e. the shaking level the structure can resist before failure. The predominant out-of-plane failure of structural wall is not a desirable failure mode due to its lower seismic capacity and drastic consequent failure of structure. The out-of-plane failure modes in a confined masonry structures can be avoided practicing lower aspect ratio of in-plane walls i.e. length to thickness ratio (to avoid horizontal bending failure mode) and height to thickness ratio (to avoid vertical bending mode), and good connectivity of wall-to-confining elements and wall-to-floor. The current construction practice in Pakistan ensure to avoid this failure mode.

The typical in-plane damage mechanisms, due to lateral loading, observed in confined masonry walls during past earthquakes and experimental studies (after Asinari, 2007; Meli et
Flexure dominated failure: it corresponds to the tensile failure of masonry and yielding of reinforcement in columns or crushing of masonry. The typical damage mechanism observed in walls with high aspect ratio (height to length ratio) and subjected to lower gravity loading. The lateral strength for this mechanism is provided by masonry tensile strength and bond strength of masonry-column interface, where ultimate strength will be provided by yielding of reinforcing steel in columns or alternatively the crushing of masonry at the compressed toe. This type of mechanism generally provide high strength and ductility relative to other mechanism. However, this type of damage mechanism is rather very rare and generally doesn’t likely in one to two storey structures. This mechanism is thus not considered for lateral strength evaluation of walls in the present study.;

Shear dominated damage of wall: it corresponds to the diagonal cracking of wall and shear failure of columns. This mechanism occur in most of the walls confined with columns of small sizes with depth less than 1.5 times the wall thickness where behavior could be similar to re-infill if the depth of column is higher (due to non-composite action of masonry and confined columns). The lateral strength for this mechanism is provided by shear and tensile strength of masonry and shear strength of confining columns. This is the most common mechanism observed during past earthquakes and experimental studies. The predominant mechanism observed in structures investigated herein is of this type (Naseer, 2009; Shahzada et al., 2011), which is thus considered for lateral strength evaluation of structural walls in the present study.;

Shear sliding damage of wall: it corresponds to the bed-joint cracking of wall and shear failure of columns. This type of failure occurs in confined walls with very low aspect ratio where the mortar strength is weak and wall is subjected to low gravity load. The lateral strength for this mechanism is provided by the shear strength of mortar and bond strength of mortar-brick interface and shear strength of columns. Due to the restriction put on the length to thickness ratio of walls to avoid out-of-plane horizontal bending, sliding mechanism is likely avoided in structures. This mechanism may be observed very rarely in most practical cases and thus is not considered for strength evaluation of walls in the present study.;

Crushing failure of wall: it corresponds to the compressed toe crushing of wall, consequently followed by the shear failure of columns. The type of failure occurs in diagonal shear dominated confined walls (where lateral load is transferred through a diagonal compression strut mechanism) with masonry of low compressive strength. This type of failure may occur in masonry of hollow concrete blocks where crushing may take place at the ends of the diagonal strut. The lateral strength for this mechanism is provided by the axial compression strength of diagonal strut. The case study structures make use of solid masonry unit. Thus, this type of mechanism is not considered for strength evaluation of walls in the present study.;

Pre-mature shear failure of columns: it corresponds to the early shear failure of confining elements due to insufficient anchorage of the vertical or horizontal confining elements. The wall in this case may be idealized as unconfined wall due to the fact that the shear capacity of confined columns reinforcement may not developed once the masonry developed its shear capacity. Thereby the strength and ductility of wall in the this damage mechanism will be much lower. This type of damage occurs in walls of poorly detailed confinement.
3.2 Simplified Modeling of Confined Masonry Wall Structures for Seismic Analysis

The study included equivalent frame modelling (EFM) technique as proposed by Magenes and Fontana (1998) and Kappos et al. (2002) among others, for unreinforced masonry structures and employed in OpenSees (McKenna et al., 2010) by Ahmad et al. (2010) for response assessment of unreinforced masonry structures. The EFM technique is extended for modelling and nonlinear time history analysis of case study confined masonry structures in the present study. In this approach, the masonry walls are idealized as one dimensional beam-column elements (wide-column analogy) with bending and shear deformation with infinitely stiff joint element offsets at the ends of the pier and spandrel elements. The beam-column element is completely defined by masonry Young modulus, shear modulus, wall sectional area and wall moment of inertia. Each element is assigned with a nonlinear lateral force-displacement constitutive law depending on the response mechanism of wall. Considering the ultimate damage mechanisms of masonry wall. Due to the provision of reinforced concrete slab, deep spandrels, bond beam above the roof and lintel level, which ensures strong-spandrel and weak-pier condition, the response of spandrel is approximately considered as elastic, whereas inelastic response is considered only in the walls. In the present study, as mentioned earlier, the diagonal shear mechanism is considered for lateral strength evaluation of confined walls. The lateral strength wall is calculated as the sum of contributions of the masonry wall and the adjacent columns (Meli et al., 2011). Figure 4 shows the damage pattern for the considered mechanism and the lateral force-displacement behavior of confined wall.

Figure 4: Diagonal shear mechanism for a confined masonry wall. From left to right: wall damage progression and the likely lateral force-displacement behavior of wall, adopted from Meli et al. (2011)

It is considered that, the capacity of columns can be reached only after the masonry has been severely cracked and its lateral resistance has significantly decreased. The shear strength model proposed by Tomazevic (1999) and Tomazevic and Klemenc (1997) is used to determine the maximum strength of wall:

\[
V_{\text{max}} = V_{d,m} + V_{s,c} \tag{1}
\]

\[
V_{d,m} = \frac{f_{maw} A_w}{C_1 b} \left[ 1 + C_2 \left( 1 + \frac{N_{aw}}{f_{maw}} \right)^{\frac{1}{2}} \right] \tag{2}
\]

\[
V_{s,c} = 0.806n a d^2 \sqrt{f_y} \tag{3}
\]

\[
C_1 = 2 \alpha b \frac{D}{H_p} \tag{4}
\]

where \( V_{\text{max}} \) represents the maximum lateral strength of confined wall; \( V_{d,m} \) represents the shear contribution from masonry for diagonal shear damage mechanism; \( V_{s,c} \) represents the
shear contribution from columns due to dowel action of reinforcement; \( f_{tu} \) represents the diagonal tensile strength of masonry; \( A_w \) represents the cross sectional area of wall; \( N_w \) represents the axial load on wall; \( b=1 \) for \( H_p/D \leq 1 \), \( b=H_p/D \) for \( 1<H_p/D<1.5 \) and \( b=1.5 \) for \( 1.5 \leq H_p/D \), where \( H_p \) represents the wall height and \( D \) represents the wall length; \( n \) represents the number of reinforcing bars in columns per wall; \( d \) represents the diameter of longitudinal reinforcing bars; \( f_c \) represents the compressive strength of concrete; \( f_y \) represents the yield strength of reinforcing bars; \( \alpha \) represents the geometric parameter associated to shape and distribution of interaction forces, proposed as 1.25. The lateral force corresponding to the cracking of masonry may be considered as 70 percent of the maximum strength whereas the lateral force corresponding to the ultimate state can be considered as 60 to 70 percent of maximum strength. The deformation limits corresponding to each limit state is generally recommended as, after Tomazevic and Weiss (2010): 0.20–0.40% for cracking limit, 0.30–0.60% for maximum strength, 2.0–4.0% for ultimate limit state. The above strength model is tested against experimentally observed shear strength value for maximum lateral strength and force corresponding to cracking in masonry wall, Figure 5 shows the comparison in terms of the ratio of experimental to calculated strength of wall, after Tomazevic and Klemenc (1997). The models seems reasonable for calculating the lateral shear strength of confined walls, however with slight overprediction.

![Figure 5: The ratio of experimental to calculated lateral strength of confined walls](image)

Figure 5: The ratio of experimental to calculated lateral strength of confined walls

![Figure 6: Lateral force-displacement hysteretic response of confined masonry wall: From left to right: force-displacement response observed during lateral cyclic test (Tomazevic and Klemenc, 1997) and simplified idealization used in the present study. In this particular case the ultimate load was observed as 40 percent of the maximum strength.](image)

Figure 6: Lateral force-displacement hysteretic response of confined masonry wall: From left to right: force-displacement response observed during lateral cyclic test (Tomazevic and Klemenc, 1997) and simplified idealization used in the present study. In this particular case the ultimate load was observed as 40 percent of the maximum strength.

The load transfer mechanism in confined wall can be idealized considering a truss system with two diagonal (Asinari, 2007; Brzev, 2007). The typical cyclic hysteretic response behavior of confined wall is characterized by a pinching behavior in which the stiffness of wall is very low in re-loading phase due to the fact that the wall has to close the previously formed gap on the tensional diagonal before it has to be stressed in compression whereas the other diagonal will relax first from the compression and will consequently form a gap due to tension.
Figure 6 shows typical cyclic response of confined wall observed experimentally and idealized employed in the present study for analysis. Similar pinching type behavior is observed also in other recent studies (Paikara and Rai, 2006; Tena-Colunga et al., 2009).

4. Fragility Analysis of Case Study Structures

4.1 Selection and Scaling of Accelerograms

In the present study, the structures are analyzed with ten natural accelerograms extracted from the PEER NGA data base with Mw 7 (0.26 std.), for stiff soil site and inter-mediate field condition i.e. sites within 15km to 30km of fault rupture (see Error! Reference source not found.). The selected accelerograms are compatible with the Pakistan building code (BCP, 2007) specified acceleration response spectrum for Type D soil and EC8 (CEN, 2004) specified response spectrum for Type C soil of NEHRP classification. The accelerograms are previously selected and employed for response assessment of masonry and concrete structures (Menon and Magenes, 2011; Pampanin et al., 2002).

Figure 7: Characteristics of selected ground motions for NLTHA: From left to right: typical shape of acceleration response spectrum with the associated uncertainties in spectral acceleration and comparison with code specified spectrum and details of individual record.
The accelerograms are linearly scaled to multiple levels of shaking intensity in order to derive structure response curve (drift demand correlated with shaking intensity) which are employed to calculate the probability of exceedance of specified limit state given the shaking intensity and derive the structure fragility functions. The fragility functions can be used to assess the performance of structures for any scenario earthquake given only the ground motion characteristic of the event (scalar-based intensity measure of ground motions). The fragility functions can be used also for economic losses the structure can incur over its design life, considering all possible earthquake that can occur and affect the structure (Ahmad et al., 2012).

The above selection and scaling procedure can provide realistic estimate of structure’s inelastic response for earthquake events of the above characteristics however due to the linear scaling procedure adopted and higher scaling factor (greater than 2), the extension of structure inelastic response to earthquake scenarios with different characteristic (e.g. different magnitude, source-to-site distance, site geology) may not provide realistic estimate where other selection options may be adopted (Bommer and Acevedo, 2004). However, recent studies have shown that all scaling and matching criteria can provide reasonable estimate of inelastic response of structure on average (Hancock et al., 2008) and even higher scaling factor may be employed (Watson-Lamprey and Abrahamson, 2006).

4.2 Structural Fragility Functions

The selected accelerograms are scaled to multiple target ground motions in order to derive the structure response curve (reported in figure 8). The present study considered scalar-based intensity measure (elastic 5% damped spectral acceleration at 0.30sec) for ground motions, which can be employed for scenario-based performance evaluation. The interstorey drift demand for each target motion is convolved with the drift capacity of the structure for various limit states in order to derive structure fragility functions, giving probability of damage exceedance for a specified shaking level.

**Figure 8:** Seismic response curves derived for case study structures using IDA. *From left to right and top to bottom:* CM1\textsubscript{Old} and CM2\textsubscript{Old} represent confined masonry structures with low material properties (existing construction) whereas CM1\textsubscript{New} and CM2\textsubscript{New} represent confined masonry structures with good material properties (new construction). The curves show the interstorey drift demand on structures for specified ground motion, represented as elastic 5% damped spectral acceleration at 0.30sec.
The probabilistic framework (Figure 3) included the integration of the joint probability density function of the drift demand and drift capacity to obtain the probability exceedance of a given damage state for specified ground motion, which is numerically inconvenient. The First Order Reliability Method (FORM) approximations (Der Kiureghian, 2005; Pinto et al., 2004) is employed in the present study to obtain the damage state exceedance probability. In the present study, the FEMA (2003) damage scale is used for the derivation of structure damage functions:

\[
\begin{align*}
\text{Target Drift Limits versus Damage Degree} \\
\text{Slight Damage:} & \quad \theta_1 = 0.7\theta_y \\
\text{(Minor cracking in masonry)} \\
\text{Moderate Damage:} & \quad \theta_2 = 1.5\theta_y \\
\text{(Major cracking in masonry)} \\
\text{Heavy Damage:} & \quad \theta_3 = 0.5(\theta_y + \theta_u) \\
\text{(Major cracking in masonry, yielding of confined column reinforcements)} \\
\text{Near-Collapse/Collapse:} & \quad \theta_4 = \theta_u \\
\text{(Extensive damage of in-plane walls, buckling of confined column reinforcement)}
\end{align*}
\]

where \(\theta\) represents the mean target drift limit states; \(\theta_y\) represents the idealized yield drift limit derived from the bi-linearization of lateral force-displacement response of wall; \(\theta_u\) represents the ultimate drift limits. The indicated damage level is attained upon the exceedance of the specified drift limit. The present study considered conservatively \(\theta_y\) as 0.30% and \(\theta_u\) as 2.0% after Naseer (2009), Shahzada et al. (2011) and Tomazevic and Weiss (2010).

Figure 9 reports the fragility functions, for all structure damage states, for case study structures designed with the material properties for existing building stock and new recommendations.

\[
\begin{align*}
\text{Figure 9: Scalar intensity-based fragility functions derived for case study structures using NDRM. From left to right and top to bottom: CM1_{Old}, CM2_{Old}, CM1_{New} and CM2_{New}. The functions can be employed for scenario-based earthquake assessment.}
\end{align*}
\]

Figure 10 reports the collapse fragility function of each case structures in terms of peak ground acceleration (PGA), in order to compare the vulnerability of each structure type. The curves show that the case study structures, CM1 and CM2, can survive ground shaking intensity of about 0.80g without collapse whereas shaking intensity great than 1.40g will cause the collapse of structures, with 95% confidence, when considering new
recommendations for the material properties whereby the ground motion level get reduced roughly by half when considering the existing material properties. The intermediate ground motions will have different level of confidence for survival and collapse of structures. It can be observed that the masonry material properties significantly affect the performance of confined masonry structures.

![Image](image.png)

**Figure 10:** Collapse fragility function for each case study structure, considering code spectrum compatible accelerograms. The plot gives the shaking intensity capable of causing collapse of the structure, with the corresponding chances of collapse.

5. Performance Evaluation of Structures for Scenario Earthquakes

The case study structures are investigated against scenario earthquake events for damage assessment. For this purpose the structures are assessed considering various earthquake magnitude and source-to-site distance, a total of 25 combinations are considered. The ground motions for each scenario event i.e. SA(0.30sec,5%), are simulated with 1000 different possibilities, considering uncertainties in ground motions, using empirical ground motion prediction equations (Abrahamson and Silva, 2008; Boore and Atkinson, 2008; Campbell and Bozorgnia, 2008). The fragility functions are employed to compute the probability of each damage state in a given scenario earthquake event.

Figure 11 shows the damage probability matrix (mean estimate of probability for each damage state) for each scenario earthquake. By and large the confined masonry structures can perform well, it can efficiently avoid the total collapse of structures in most of the larger ground motions. However, at close source-to-site distance and larger magnitude earthquakes significant chances are observed for the collapse of structures in case of structures of existing characteristics whereas confined structures provided with good quality of masonry material can perform well.
5.1 Summary, Conclusions and Future Developments

5.1.1 Summary

The aim of the study was to assess the seismic performance of confined masonry construction schemes subjected to strong ground motions. The study included analytical investigation of two cases study low-rise (two storey) confined masonry structures designed with the existing building stock material properties (low quality material) which is subsequently designed with the material properties as per new recommendations (good quality material). The recent investigations carried out on the constituent material, masonry panels, walls and reduced scale models are considered for the mathematical modelling and nonlinear seismic analysis of case study structures. The structures are analyzed using a suite of ten natural accelerograms and nonlinear incremental dynamic analysis technique. The structures fragility functions are derived considering various level of damage e.g. slight, moderate, heavy and collapse, in the system. For both good quality and low quality building material, the ground motions capable of causing collapse of the structure are obtained with the associated chances of exceeding collapse in a specified shaking intensity. It can help identify the shaking intensity capable of causing collapse of the structure, with different confidence level. Furthermore, the case study structures are tested against simulated earthquake scenarios. The study shows that, the typical confinement of masonry walls can avoid the total structural collapse in most of the strong ground motions ensuring occupant’s safety, however the damages to structures in large earthquake events is significant, where repair will be required in the form of cement grouting and/or reinforced plastering of masonry walls and repair of confining elements. The study shows that besides the good behavior offered by the confinement scheme, the role of construction material’s mechanical properties in performance improvement is significant i.e. confined masonry construction using good quality building material (as mentioned) can provide tremendous resilience against earthquake induced strong ground motions.
5.2 Conclusions

The following conclusions are derived based on the seismic analysis of typical low-rise (two storey) confined masonry structures practiced in Pakistan. The structures are considered with in-plane lateral response to ground shaking and shear dominated damage in masonry confined walls i.e. diagonal cracking of masonry wall followed by the shear failure of confining columns. The lateral strength of in-plane walls in the system is provided by the composite action of masonry and reinforced concrete confining columns. The nonlinear response of confined wall is defined by a trilinear lateral force-displacement response with pinching hysteretic behavior. The findings may be extended to confine masonry structures of the characteristics as mentioned.

1. Confined masonry structures provided with wall density of 4.20% to 4.60% in the direction of earthquake excitation and constructed using good quality building material can avoid the collapse of structures in large earthquakes capable of causing shaking intensity event up to 0.80g whereas the shaking intensity of 1.40g and above can cause the collapse of structures, with 95% chances.

2. Confined masonry structures of the above characteristics constructed using relatively low quality building material can avoid the collapse of structures in earthquakes with shaking intensity up to 0.40g whereas the shaking intensity of 0.70g and above can cause the collapse of structures, with 95% chances.

3. The seismic capacity of confined masonry structures may be reduced by 50% if low quality construction material is employed for the construction of masonry wall.

4. Investigating on the seismic behavior of structures in scenario earthquakes show that the typical confinement of masonry walls ensure excellent performance of structures against earthquake induced ground motions, avoiding the structural collapse in most cases thereby ensuring safety of the occupants in even in larger earthquake events.

5. In most of the test scenario earthquakes, the structures are found with slight to moderate damages in the form of cracking (more or less extensive) in masonry structural walls which may require repair in the form of cement grouting to restore the structure’s lateral stiffness and strength. Due which the structures may cause significant economic losses in regions with moderate to high frequent seismicity.

6. Confined masonry structures of the above characteristics constructed in good quality material can be confidently practiced in the highest hazard zones (Zone 4, PGA ≥ 0.40g) of Pakistan.

7. Existing confined masonry structures (low quality masonry material) of the above characteristics can be confidently used in high hazard zones and lower i.e. (Zones, PGA < 0.40g). These structures may not be used confidently in Zone 4 (PGA ≥ 0.40g).

8. A ten percent increase in the wall density (when wall length is increased only) didn’t cause appreciable difference in the seismic performance due to the reason that the better performance is governed by the deformation capacity (ductility) of the system.

5.3 Future Developments

1. Investigation is required on full scale confined masonry walls with various geometries and loading in order to further improve/validate the available shear strength models,
develop force-deformability constitutive law for walls and better understand the nonlinear hysteretic behavior of confined walls for the design and analysis of structures.

2. Investigation is required to quantify and relate the seismic resilience of confined masonry structures with basic masonry material properties. It can provide help on identifying critical ground motions for a specified structures and which can in turn provide help on the minimum requirements for design of structures given the site seismicity.

3. Confined masonry structures constructed in accordance with the current recommendations can perform tremendously better during strong ground shaking however the damages in these structures are significant which may require regular repair. In this regard, study is required on the estimation of economic losses the structure can incur over its design life due to regular repair.

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