Seismic Performance Evaluation of Reinforced Plaster Retrofitting Technique for Low-Rise Block Masonry Structures

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Abstract: Low-rise unreinforced block masonry structures are investigated for resistance against site amplified ground motions, before and after retrofitting with reinforced plaster. The aim of the study was to evaluate the effectiveness of indigenous and cost-effective retrofitting technique for improving the seismic performance of ordinary masonry structures. The retrofitting technique involved cement grouting of damaged walls and fixing steel wire mesh on wall surface, plastered with rich cement mortar. Experimental investigations carried out on block masonry material and prototype walls are analyzed to retrieve the mechanical characteristics for mathematical modeling of structures. Two case study low-rise (two storey) structures are assessed through nonlinear incremental dynamic time history analysis technique using natural accelerograms. The considered retrofitting technique increases the lateral load carrying capacity of in-plane walls by about 10 percent and ductility capacity by 100 percent and ensures flexure (toe crushing) response mechanism to lateral loading. The technique increased the seismic resistance of structures to ground shaking by 43 percent. It is observed that the retrofitted structures can survive ground shaking intensity up to 0.40g/0.50g whereas shaking intensity of 0.73g/0.90g (and above) may cause the structure collapse (with 95 percent confidence), indicating that the technique can be confidently used for improving the seismic performance of block masonry structures.

Keywords: block masonry structures; reinforced plaster; ferrocement; cost-effective retrofitting, structure useful life.

1. Introduction:

Significant proportion of building stock in urban areas are constructed using concrete block masonry in cement-mortar. For example, concrete blocks are used in construction of walls in the order of 10-20 percent in most of the Districts of Punjab and Sindh Provinces of Pakistan. In few other cases the percentage can reach event up to 40-60 percent e.g. District Karachi in Sindh and few other Districts in Northern areas of Khyber Pakhtoonkhwa of Pakistan (ERRA, 2006; Maqsood and Schwarz, 2008). Concrete block masonry is used mostly for one to two storey residential structures in urban areas. However, concrete block masonry walls are also used as an infill panel in reinforced concrete frame construction e.g. the City like Karachi (Badrashi et al., 2010) among others. Also, recently following major earthquake in Pakistan, the use of concrete block units has been increased for the re-construction activity in major affected areas (both urban and rural) due to the fact that concrete blocks can be easily and rapidly produced using even small machines (Stephenson, 2008).

In the recent major earthquake Mw 7.6 2005 Kashmir, about 30 percent of the building stock subjected to ground shaking included block masonry structures in the nearby urban areas. Almost 60 percent of block masonry structures were either heavily damaged are collapsed (Naseer et al., 2010; Rossetto and Peiris, 2009), which is significant. The recent investigation on the seismic risk assessment of existing structures in Pakistan reported that regional block masonry structures subjected to ground shaking intensity of 0.30g can result in the collapse of 70 percent of the exposed building stock (Ahmad, 2011), which is due to the extremely low mechanical characteristics of existing block masonry material. This means that existing block masonry construction is at risk in the high hazard zones of the country e.g. Zone 3 (0.24g–0.32g) and Zone 4 (> 0.32g) as specified by BCP (2007), indicating that ordinary block masonry structures cannot be confidently constructed in high hazard zones. Similar, findings have been reported also by Mahmood and Ingham (2011) however observing also that significant amount of block masonry typologies are earthquake-prone even in moderate hazard zones e.g. Zone 2B (0.16–0.24g). This situation motivate research on the seismic vulnerability.
assessment of individual block masonry structures toward disaster risk reduction (saving life during strong ground motions) using efficient, yet affordable, retrofitting techniques.

Cost-effective and efficient retrofitting techniques are essential, particularly for the developing part of the world exposed to strong ground motions. Retrofitting techniques for masonry structures are evolved to a much mature stage (ElGawady et al., 2004), however its design and application may still be affected by many factors like the structures typology and its existing condition, performance objective, cost of retrofitting, available time, resources and materials, preservation of historical value etc., particularly in the developing part of the world e.g. Pakistan, India, Iran, among others which predominantly include masonry structures. This clearly indicates the importance of research on the investigation of efficient and cost-effective retrofitting techniques for masonry structures.

The paper thus presents investigation carried out on low-rise block masonry structures practiced mostly in the urban areas of Pakistan, for the seismic resistance evaluation of existing structures and their possible improvement using indigenous and cost-effective retrofitting technique. It included seismic performance evaluation of structures; existing and retrofitted (damaged structures) with reinforced plaster technique developed and practiced at the Earthquake Engineering Center of Peshawar. Experimental investigations carried out recently at the Earthquake Engineering Center and University of Engineering and Technology (UET) Peshawar (Shoaib et al., 2011) on block masonry assemblages and prototype walls (before and after retrofitting) are considered for design and mathematical modelling of case study structures.

Nonlinear dynamic time history analyses of four case study structures are performed using natural accelerograms, where the structures are assessed using probabilistic approach taking into account the variability of ground motions in performance evaluation. Ground motions with different exceedance probability is obtained for specified performance level of structures. Example structures are investigated for the seismic performance evaluation of the retrofitting technique considering a single scenario earthquake and possible events over the design life.

**Masonry Walls Retrofitting Technique (Reinforced Plaster):**

The retrofitting technique investigated in the present study included grout injection of cracks (for already damaged walls) and application of steel welded wire mesh on the surface of walls, plastered with rich cement-sand mortar, the so called ferrocement overlay: a thin layer of reinforced concrete consists of closely spaced small size wire-mesh completely embedded in cement mortar (ACI 549R-97, 1997). The technique require relatively low level technical skills for fabrication and application of ferrocement. The provision of closely spaced steel wire-mesh in the ferrocement overlay provides significant tensile strength to retrofitted walls with low economic cost, without adding significant additional mass to the structural system, which makes the technique highly appealing in developing part of the world exposed to significant earthquakes.

1.1 **Reinforced Plaster Retrofit Design Scheme:**

In the first phase a pre-fabricated wire mesh is fixed on the wall surface through fixing screws, where the screws are fastened in the plastic plugs inserted in the wall. Holes are formed in masonry units for inserting plugs. This technique is recently developed and practiced at the Earthquake Engineering Center of Peshawar. Figure 1 depicts the schematic presentation of reinforced plastering of existing masonry walls. For the case of already damaged walls, the grouting of cracks is also needed to be performed before plastering mesh-fixed wall.

Recently, Shoaib et al. (2011) investigated this retrofitting technique experimentally for the performance improvement of damaged block masonry prototype walls. In this case, the grouting of cracks is performed 24 hours after the plastering of mesh-fixed wall. For this purpose, nozzles (10mm diameter and 80mm length) were pre-inserted in the cracks at a distance of 300mm to 500mm along the cracks soon after fixing the wire mesh, where grout is injected to the cracks with pressure pump. The steps involved in grouting technique were outlined previously by Calvi and Magenes (1994), Hamid et al. (1994) and Schuller et al. (1994). The grouting injection material included a cement-based grout comprised of 10 parts of Portland cement, one part of lime and ultra expansion agent employed at a rate of 250 grams per 50 kilograms of cement (in the proportion of 200:20:1) with a water cement ratio of 0.8. The wire mesh was designed of a 12.70mm x 12.70mm galvanized steel mesh made of 1mm diameter wires having 200 Mpa tensile strength. The mesh is fixed to the wall surface by 40mm long No.10 screws and plastic Ravel plug, provided as two plugs per 100 square centimeter, called as 18 gauge wire mesh. The plaster consisted of cement-sand mortar which is prepared in proportion of 1:4 and applied in average thickness of 20mm.
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Figure 1: Schematic of Reinforced Plaster Retrofitting Technique for Existing Masonry Walls Developed and Practiced at the Earthquake Engineering Center of Peshawar. From Left to Right and Top to Bottom: Required Accessories, Drilling for Inserting Screw Plugs, Application and Fixing of Wire Mesh and Plastering of Wall (Shahzada, 2006)

1.2 Observed Behavior of Block Masonry Walls (Before and After Retrofitting):

This section is largely based on the experimental study carried out by Shoaib et al. (2011) on block masonry material and walls and the reinforced plaster retrofitting technique recently developed and practiced by the Earthquake Engineering Center of Peshawar. The following sections briefly describe the testing program, the mechanical characteristics of block masonry and in-plane response of case study block masonry walls (before and after retrofitting) important within the scope of present research study.

1.2.1 Experimental Program:

The experimental investigation included in-plane quasi-static cyclic test on six masonry walls: three sample walls were representative of a short pier in single storey structures (and first floor of double storey structures), other three sample walls were representative of a ground floor short pier in double storey structures. The piers were first tested in the unreinforced state which were retrofitted and tested again. The tested walls were deformed into life safety performance range till the peak load was developed, strength and stiffness degradation is noticed and when repair to damage was feasible.

Table 1: Material Properties of Block Masonry Walls Tested at UET Peshawar, after Shoaib et al. (2011)

<table>
<thead>
<tr>
<th>Property</th>
<th>Observation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Block (Solid) Dimensions</td>
<td>150mmx200mmx300mm</td>
</tr>
<tr>
<td>Mortar Composition</td>
<td>1:6 (Cement-Sand)</td>
</tr>
<tr>
<td>Block Compressive Strength</td>
<td>3.97 MPa</td>
</tr>
<tr>
<td>Mortar Compressive Strength</td>
<td>3.60 MPa</td>
</tr>
<tr>
<td>Masonry Compressive Strength</td>
<td>2.23 MPa</td>
</tr>
<tr>
<td>Diagonal Tensile Strength</td>
<td>0.20 MPa</td>
</tr>
<tr>
<td>Water Absorption</td>
<td>8.60%</td>
</tr>
<tr>
<td>Block Modulus of Rupture</td>
<td>1.93 MPa</td>
</tr>
<tr>
<td>Friction Coefficient</td>
<td>0.378</td>
</tr>
<tr>
<td>Cohesion</td>
<td>0.10 MPa</td>
</tr>
<tr>
<td>Modulus of Elasticity</td>
<td>2375 MPa</td>
</tr>
<tr>
<td>Shear Modulus</td>
<td>47 MPa</td>
</tr>
</tbody>
</table>
1.2.2 Basic Mechanical Properties of Block Masonry:

It also included tests on masonry prism and masonry panels, three samples for each, for the estimation of compression, shear and diagonal tension strength, Young and shear elastic moduli. Table 1 reports the basic mechanical properties of block masonry investigated in the present research study.

1.2.3 Observed Response of Walls (Before and after Retrofitting):

All of the walls have shown a flexure response at the initial stages whereby cracks where developed at the base of the wall which propagated with increasing lateral displacement till rocking of the wall is initiated. However, inclined shear cracks were also developed before pure rocking of wall may be achieved. Sliding is also observed at the bed joints of wall. The final damaged wall included severe inclined cracks in masonry units and cracks at the head and bed joints. The retrofitted walls developed flexure cracks at the base which is propagated to ensure rocking of the wall. Significant inclined cracks were observed at the compressed toe of wall, which could finally lead to toe crushing and possibly simultaneous overturning of the wall.

Performance-Based Damage Scale:

\[
\begin{align*}
\text{Slight Damage:} & \quad \theta_1 = 0.7\theta_y \\
\text{Moderate Damage:} & \quad \theta_2 = 1.5\theta_y \\
\text{Heavy Damage:} & \quad \theta_3 = 0.5(\theta_y + \theta_u) \\
\text{Near-Collapse/Collapse:} & \quad \theta_4 = \theta_u
\end{align*}
\]

For all the walls, the retrofitting technique changed the response mechanism from a flexure-shear mixed type response to a flexure-toe crushing type response, when reinforced plaster was applied on both faces of the wall. The mechanism is not changed when reinforced plaster was applied on one face of the wall only. Shoaib et al. (2011) reported the lateral force-displacement response of tested walls (before and after retrofitting), where it was observed that the retrofitting technique can increase the lateral strength by ten percent and increases the ductility (the ratio of ultimate displacement capacity to yield displacement) by 100 percent due to ensuring flexure response of walls. Increase in the idealized yield displacement capacity of the retrofitted wall relative to the counterpart unretrofitted wall was not significant.

In the present study, the following damage scale is developed (after FEMA, 2003) in view of the trend towards deformation-based seismic design and assessment of block masonry structures (before and after retrofitting).

\[
\begin{align*}
\theta_i & \text{ represents the mean drift limit states i.e. drift threshold value; } \theta_y & \text{ represents the idealized yield drift limit derived from the bi-linearization of lateral force-displacement response of wall; } \theta_u & \text{ represents the ultimate drift limits. The indicated damage state and performance levels are attained upon the exceedance of the specified drift limit. The approximated observed } \theta_y & \text{ is 0.11% for unretrofitted wall and 0.13% for retrofitted wall while } \theta_u & \text{ is 0.62% for unretrofitted wall and 1.25% for retrofitted wall.}
\end{align*}
\]

2 Example Structures for Investigation:

2.1 General Characteristics of Block Masonry Structures in Pakistan:

The block masonry construction in Pakistan consists of load-bearing walls of 150mm to 200mm thickness, using single block unit (block size 300mm length, 200mm width, 150mm thickness) i.e. single-leaf, constructed in cement-sand mortar mix ratio of 1:6. The blocks are generally manufactured from combination of cement, sand, and crushed stone with a mix proportion (by volume) of 1:6:12 to 1:8:14. This mix proportion results in 80 to 100 block units for 50 kilograms of cement (one bag) with significantly low compressive strength (Naseer et al., 2010). However, the current recommendations for reconstruction suggest block units of compressive strength more than 10 MPa, which cannot be achieved looking at the current practice in the field.

These structures are provided with 130mm to 150mm thick reinforced concrete slab, as also common for brick masonry construction type in urban exposure. Light wooden and steel roof truss with Galvanized Iron sheet and wooden floors are also practiced now in rural and urban parts of the country. The clear inter-story height and ground floor height ranges roughly between 2.0m to 3.0m and 2.5m to 3.5m, respectively. The current construction guidelines recommend the use of band beams at plinth/ lintel/ roof levels in order to reduce the aspect ratio of wall, avoid the out-of-plane bending of walls and ensuring in-plane integrity of structures.
2.2 Example Structures: Geometric Detailing and Mathematical Modelling for Analysis:

Figure 2 shows typical plan of structures analyzed in the present research study. Such construction practice can be found in the major seismic areas of Pakistan e.g. the northern parts of the country. However the current construction recommendation i.e. symmetric detailing for construction to avoid the torsional inertia in the structures subjected to ground motions, is followed for case study investigation.

In the present investigation, one of the structures (left) is considered with floor area of 50 square meter and wall density of 4.20 percent in the excitation direction, while the second with floor area of 75 square meter and wall density of 4.60 percent. Since for a given masonry typology, these parameters significantly influence the response of structures (Ahmad et al., 2011a) in which the wall density has relatively high impact on the seismic performance of structures.

The equivalent frame modelling technique proposed by Magenes and Fontana (1998) and Kappos et al (2002) is used for the mathematical modelling and nonlinear dynamic time history analysis (NLTHA) of case study structures. This technique can be efficiently extended for vulnerability assessment of structures using fully dynamic approach. In the present study, the analysis is carried out using OpenSees (McKenna et al., 2010).

In this approach, the masonry walls are idealized as one dimensional beam-column elements 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, after Calvi and Magenes (1997) and Tomazevic (1999); the diagonal shear, flexure (toe crushing), and shear sliding analytical models are employed for lateral strength evaluation of a given wall:

\[ V_f = \frac{f_{tu}D t}{b} \left(1 + \frac{P}{f_{tu}}\right) \frac{1}{2} \]  

\[ V_d = \frac{\rho D^2 t}{2H} \left(1 - \frac{P}{f_{tu}}\right) \]  

\[ V_s = \frac{D t (1.5c + \mu p)}{1 + 3.25H_p \frac{pD}{D}} \]  

where \( V_f \) represents the lateral strength for flexure (toe crushing) mechanism; \( D \) represents the length of wall; \( t \) represents the thickness of wall; \( P = P/(Dt) \) represents the mean vertical stress due to axial load \( P \); \( H_p \) represents the height of wall; \( \psi \) is 1.0 for a cantilever pier and 0.5 for a pier with fixed-fixed boundary conditions; \( f_{tu} \) represents the compressive strength of masonry; \( k \) is a coefficient used to idealize the stress distribution at the compressed toe of wall (Magenes et al., 2000), considered as 0.85; \( V_d \) represent the shear strength for diagonal shear damage mechanism; \( b \) is the diagonal tensile strength; \( b=1 \) for \( H_p/D \leq 1 \), \( b=H_p/D \) for \( 1<H_p/D \leq 1.5 \) and \( b=1.5 \) for \( 1.5 \leq H_p/D \); \( V_s \) represents the sliding shear strength; \( \mu \) and \( c \) represent the coefficient of friction and cohesion of masonry as global strength parameters.

A correction factor is proposed (Magenes and Calvi, 1997; Mann and Muller, 1982) to transform the local parameters obtained using triplet/couplet test to global parameters. The above shear strength models have been discussed earlier in Magenes and Calvi (1997) and Tomazevic (1999) for appropriate applications.

Due to the provision of reinforced concrete slab, deep spandrels, ring beam above the wall and band beam at lintel level (in order to ensure strong-spandrel and weak-pier condition), the response of spandrel is approximately considered as elastic, whereas inelastic response is considered only in the piers.

Figure 3 shows the comparison of lateral shear strength obtained experimentally and calculated using the above analytical models. The models shows reasonable performance in terms of providing estimate of lateral strength and predicting response mechanism. In case of retrofitted walls the increase in masonry compressive strength (24%), diagonal tension and shear strength (74%) due to retrofitting are considered accordingly in the strength evaluation.

In case of shear sliding model the friction coefficient component is not increased due to the fact that experimental data is not available to define \( \mu \) for reinforced plastered wall. Due to this the shear sliding model will result in lower estimate of lateral strength for retrofitted wall. This situation also hinders the reliable application of sliding shear model (modified Mohr-Coulomb shear model). However from the experimental tests, it is observed for the considered retrofitting technique that, although diagonal shear cracks may occur in wall, toe crushing is always the governing mechanism. Thus, only the diagonal shear and toe crushing models are used confidently for retrofitted wall in the present study.

The indicated increase in the mechanical properties of masonry due to wire-mesh application is typical for the considered reinforced plaster design scheme. Thus, generalization of lateral strength model (considering the modified mechanical properties) to retrofitted walls with different geometry is reasonably possible. However, it is worth to mention that the effect of reinforced plaster on the mechanical characteristic depend significantly on the strength and thickness of mortar used for plastering, gauge spacing of wire mesh (the amount of reinforcement), mesh orientation, the quality of bond at the interface of reinforced plaster and wall and spacings of connecting screws (ACI 549R-97, 1997; ElGawady et al., 2004; Shah, 2011).
unretrofitted case, as recommended by Ahmad et al. (2011b) for shear mechanism of masonry walls. Similar, constitutive law is also used recently by Menon and Magenes (2011) for seismic analysis of masonry structures. The retrofitted walls showed flexure response with toe damage for which bi-linear rocking re-centering rule is approximately used. Figure 4 shows typical force-displacement rule for shear and flexure mechanism of masonry walls which are employed for unretrofitted and retrofitted cases, respectively, in the present study.

Figure 4: Force-Displacement Constitutive Law for In-Plane Lateral Response of Walls, after Ahmad (2011b):
From Left to Right: Force-Displacement Idealization for Shear Mechanism (Approximately Extended to Unretrofitted Walls) and Force-Displacement Idealization for Flexure Mechanism (Approximately Extended to Retrofitted Walls)

3 Performance Evaluation of Block Masonry Structures:

3.1 Nonlinear Dynamic Reliability Based Seismic Assessment Method (NDRM):

The study included a nonlinear dynamic reliability based approach for probabilistic-based seismic performance assessment of case study structures. This includes the incremental dynamic analysis (IDA) technique for structural analysis as proposed by Vamvatsikos and Cornell (2002), to obtain demand (drift demand) on a structure in a probabilistic fashion which is convolve with the probabilistic capacity (drift capacity) to obtain the exceedance probability of specified damage state (e.g. collapse). Figure 5 describes graphically the probabilistic-based assessment framework.

Generally, the exceedance probability of specified damage state for a given ground motion is obtained using the classical reliability formulation that includes the integration of joint probability density function of demand and capacity, which is nevertheless inconvenient to evaluate numerically. The present study thus employed the First Order Reliability Method FORM approximations (Der Kiureghian, 2005; Pinto et al., 2004) to obtain the damage state exceedance probability. This procedure has been recently investigated against moderate and large magnitude earthquakes for damage and collapse assessment of existing structures in Pakistan (Ahmad, 2011) which shows reasonable performance of the approach.

The case study structures are analyzed dynamically using NLTHA with ten natural accelerograms extracted from the PEER NGA data base for stiff soil site and inter-mEDIATE field condition i.e. sites within 15km to 30km of fault rupture (see Figure 6 and Table 2).

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 linearly scaled to multiple levels of shaking intensity in order to derive response curve (drift demand correlated with shaking intensity) for a given structure which are employed to calculate the probability of exceedance of specified limit state given the shaking intensity.

3.2 Collapse Assessment of Case Study Structures:

In the present study the collapse limit state corresponds to the damage level when structure is forced to near-collapse or complete collapse state. This damage level occurs when collapse prevention (CP) is exceeded as specified for the performance-based assessment and design of structures. The physical damage corresponds to the state of structure on the verge of partial and total collapse where the occupants may be injured and structural damages are not economically repairable.
In the present study, the tested unretrofitted walls were deformed into life safety performance range till the peak load was developed, strength and stiffness degradation is noticed and when repair to damage was feasible. This means that the wall was pushed in to life safety performance range but the CP limit state was not passed. The ductility and drift limit computed in this case is lower than the actual CP limit states but which is nevertheless conservative for assessment purpose. The retrofitted walls were also deformed into the life safety performance range while CP was not reached, however significant plasticity was observed in the wall. Also, significant damage was observed at the compressed corners which may lead to toe crushing and possibly simultaneous overturning of the wall.

Thus, the ultimate drift capacity used in the present study is a conservative (relatively lower than the actual) estimate for CP limit state. Thereby the structures are presumably considered with reduced ultimate drift capacity, which is nevertheless conservative.

Figure 7 shows the response curve, which shows the interstorey drift demand for specified level of shaking intensity, derived through IDA procedure. For each of the target PGA, the drift demand is convolve with drift capacity using the FORM approximation. The structure
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reliability against the collapse is calculated (shown in Figure 8) which can provide help on identifying the critical ground motions with different confidence level.

The reliability curves show that the case study structures (UBM1, UBM2, UBM1R and UBM2R) can survive ground shaking intensity of 0.28g, 0.35g, 0.40g and 0.50g respectively whereas ground motions with shaking intensity greater than 0.55g, 0.73g, 0.73g and 0.90g respectively will cause the collapse of structures (with 95% confidence). The intermediate shaking intensity will have different level of confidence for survival and collapse of structures.

This indicates that similar block masonry structures retrofitted with reinforced plaster can be used confidently in high hazard zones of Pakistan like Zone 3 and Zone 4 specified by building code BCP (2007), in order to ensure the safety of occupants during strong ground shaking. The curves also show that the seismic capacity of block masonry structures can be significantly increased with increasing wall density. For example a ten percent increase in the wall density increased the seismic capacity by 25%, as observed in the capacity of UBM2 relative to UBM1 structure.

3.3 Design Life Performance of Block Masonry Structures:

The above discussion mainly considered the collapse capacity of structures against earthquake induced ground motions, considering the performance of structures for a single earthquake scenario having magnitude of about Mw 7 (0.26 std.) within 15km to 30km of fault rupture. Also, the above discussion is limited to the performance of structures to ensure the safety of occupants during a specified earthquake events. Considering the fact that a structure may be subjected to various frequent and rare event earthquakes in its design life, which may cause different level of damage in the structures due which repair will be required for the structure at regular interval. This situation also make necessary to assess the performance of a structure for the intended economic losses it can incur over the design life.

For this purpose fragility functions are derived for case study structures (Figure 9) using the procedure outlined.
in Ahmad (2011). Vulnerability curves are derived convolving the structure fragility functions with the economic consequence factor (ECF), where ECF relates physical damage with the monetary loss for the required repair as a fraction of total replacement cost of structure.

The damage scale, as developed earlier, and ECF proposed by FEMA (2003) is used in the present study. This damage scale requires the CP and Yield limit state drift capacities to develop the whole damage scale, see Eq. (1) to (4). The FEMA ECF specify repair cost ratio of 0.02 for slight damage, 0.10 for moderate damage, 0.50 for heavy damage and 1.0 for near-collapse/collapse structures.

For this purpose all possible seismic sources around the region is considered, for which possible ground motions are generated using empirical ground motion prediction equations. The ground motions are assigned with the associated annual probability in order to derive the hazard curve using the standard PSHA framework (see Figure 11). In the present study three ground motion prediction equations (Abrahamson and Silva, 2008; Boore and Atkinson, 2008; Campbell and Bozorgnia, 2008) and two seismicity models (GSHAP after Giardini et al., 1999; and PMD, 2007) are used to derive hazard curve. The vulnerability and hazard curves are convolved to estimate the AAL. Figure 12 shows the AAL calculated for each of the case study structures.

It is observed that UBM1, UBM2, UBM1R and UBM2R structure will acquire AAL of 1.60, 0.97, 0.67 and 0.38 percent of the replacement cost of structure respectively, resulting in 63, 103, 150 and 263 years of useful life of structures. This indicates that the retrofitting technique will reduce the economic losses of both case study structures by 60 percent on annual bases and will increase the useful life of structure by about 150 percent. The useful life of structure here represents the time span when the structure accumulated economic losses reaches the replacement cost.
4 Closure:

4.1 Summary:
The aim of the study was to evaluate the performance of block masonry structures before and after retrofitted with reinforced plaster in order to assess the feasibility of cost-effective and indigenous retrofitting technique for improving the seismic performance of masonry structures. Investigation is carried on block masonry construction system against earthquake induced site amplified ground motions. It included incremental dynamic analysis of example structures for the derivation of seismic response curves which are analyzed through FORM approximation in order to derive the structure reliability curves. Furthermore, fragility functions are also derived which are employed to derive the vulnerability curves. Additionally, PSHA is carried out to develop seismic hazard curves for Manshera District (a high seismicity region). The economic loss the structure can incur on annual bases is calculated convolving the site hazard with the structure vulnerability curves. This also gave, inversely, the useful life of unretrofitted and retrofitted structures. The investigation showed significant good performance of the reinforced plastering technique in improving the capacity of block masonry structures against earthquake induced shaking.

4.2 Conclusions:
The following conclusions are derived based on the seismic analysis of block masonry structures that can ensure in-plane lateral response to ground shaking (right through the length of walls). The out-of-plane failure of walls in these structures are avoided practicing restricted wall aspect ratio (height-to-thickness ratio) and ensuring good wall-to-wall and wall-to-floor connectivity. The case study structures are considered with rigid reinforced concrete floors provided with ring beam (monolithically connecting walls to the floor), band beam (at the lintel level) and deep spandrels (ensuring strong-spandrel to weak-pier condition). The in-plane walls are considered with inelastic response while the spandrels are considered with elastic response. The study considered the experimentally obtained mechanical properties of block masonry in Pakistan. In the case study structures, the lateral strength of unreinforced wall is provided by shear damage mechanism (sliding shear model, modified Mohr-Coulomb strength model) while the counterpart retrofitted wall provided lateral strength with flexure (toe crushing) mechanism. The lateral strength of retrofitted wall increased by ten percent while the ductility and drift capacity increased by about 100 percent. These findings are also applicable to block masonry structure of the above characteristics and the reinforced plaster design scheme as mentioned.

- For both unretrofitted and retrofitted walls, the analytical lateral shear strength models are comparable in predicting the response mechanism of walls investigated though in-plane quasi-static cyclic test. The models (Mohr-Coulomb shear model in case of unretrofitted wall and toe crushing model in case of retrofitted wall) provided reasonable estimate of lateral shear strength.
- The investigation on case study structures showed that considering the survival of structures against collapse, the technique increases the seismic capacity by 43% which can provide significant increase in the ground shaking intensity the structure can survive (with 95% confidence).
- Locating and investigating the structures in high seismicity area showed that the technique will reduce the economic losses by about 60 percent on annual bases and will increase the useful life of structure by about 150 percent.
- The example unreinforced block masonry structures retrofitted with reinforced plaster can ensure the safety of occupants during strong ground motion, thus can be confidently used in moderate to high hazard zones, as mentioned.
- The better seismic performance of case study retrofitted structures is primarily governed by the increase in ductility capacity of walls while less contribution (almost negligible) is provided by ten percent increase in lateral strength.
- Additionally, the investigation also showed that a ten percent increase in the wall density increases the seismic capacity by 25 percent, reducing the economic losses by 40 percent and increasing the structure useful life by 63 percent. This indicates that the wall density of masonry structures is also a crucial parameter in improving the seismic performance.

4.3 Future Developments:
- There is a need to further investigate the reinforced plaster retrofitting technique for masonry walls of different geometry and loading condition. It can help in developing generalized lateral shear strength models which can be employed then confidently to assess the performance of retrofitted structures analytically.
- The present study investigated masonry structures with toe crushing mechanism of retrofitted walls and ductility 100 percent greater than the counterpart unreinforced case. Further investigated is required to understand the response mechanism and ductility capacity for various cases of walls varying the geometric and loading conditions to generalize the effect of reinforced plaster for structures with any geometric detailing.
- There is a need to further investigate retrofitted walls with various geometries and reinforced plaster
design schemes to deduce deformation-related performance objectives, defining different levels of required repair with the associated cost of repair, in retrofitting techniques in view of the trend towards deformation-based seismic design of structures.

- The present case study investigation considered bi-linear hysteretic rule for shear damage mechanism (unretrofitted case) and bi-linear centering rule for toe crushing mechanism (retrofitted case). However, investigation is required in this regard which may further improve the findings reported herein and which can in turn increase the confidence of approach in future applications.

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