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# Confined and unreinforced masonry structures in seismic areas: Validation of macro models and cost analysis

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ABSTRACT

Modern structural design requires consideration of sustainability parts from a life cycle perception, but also the initial design phase in which seismic actions have a substantial influence on the design of the structure. In recent times, the seismic assessment of masonry buildings by means of macro-element modeling methodologies has become popular, by application of performance-based evaluation techniques using nonlinear lateral load procedures (Pushover). This study addresses the endorsement of these methodologies by referring to two full-scale brick masonry structures subjected to a lateral loading conditions. The lateral load response of tested unreinforced masonry (URM) and confined masonry (CM) structures is compared with the response of the numerical models. The considered numerical models have good agreement for satisfactorily predicting the response of the experimental test and hence are capable of being used in a performance-based evaluation. Then, pointing to the characteristic housing of northern Pakistan and its typical design with a reinforced concrete (RC) building, the validated numerical models are used to estimate the hazard-resistant potentials of the URM and CM options for one, two and three story options, particularly in relation with maximum lateral load capacity. The load deformation response of both the typologies was also compared for the mentioned three story levels. It was observed that by confining the masonry its ductility capacity increases considerably, hence making it more suitable to be used in earthquake prone regions. Masonry structures are also compared regarding the construction costs compared to the RC typology. With regard to the dwellings studied, the projected lateral load behavior for masonry structures indicate the ability to withstand lateral loads adequately. These structures also allow a significant reduction in costs (up to 28%) compared to RC, hence appearing as challenging alternatives.

## 1. Introduction

The building construction has a great effect on the total budget of a family. Furthermore, people live the maximum part of their lives in the buildings. Low rise structures (three story or less) are the common type for houses, demanding thus specific consideration in the development of workable solutions for their construction. The accepted construction solution indicates itself as an essential primary investment for house construction and is taken as the emphasis of this study.

Structures are supposed to offer safety to residents. The incidence of strong seismic events (e.g., 2005 earthquake of Kashmir), emphasized the significance of structures designed below par for seismic loads regarding: casualties, fatalities, post-event disturbances, repair costs and repair time [1]. It is recognized that seismic hazards can occur anywhere in the world producing huge losses. Therefore, the effect of these seismic forces needs to be effectively considered in the structural design, as determined in latest practices and seismic safety codes for evaluation of buildings, e.g. [2–4]. Economical construction methods can result in greater vulnerability to earthquake forces, like masonry structures in comparison with the reinforced concrete (RC) structures. For this reason, the latter has developed in to the leading construction approach around the world, even in the case of small dwellings situated in the low seismicity areas. However, in several conditions and considering the seismic response, the unreinforced masonry (URM) or confined masonry (CM) structures might be otherwise considered as a construction alternative for low-rise structures [5].

The safeguarding of existing building infrastructure in seismic areas necessitates, thus, a tactical plan and appropriate procedure containing features such as: Phase 1—primary examination, understanding of documentation and the context, and assessment of the common

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structural features of the building; Phase 2—complete diagnosis, using certain non-destructive testing and numerical analyses; Phase 3—designing of structural reforms, as per safety requirements, upgrading, retrofitting and strengthening, as well as the contemporary codes of action such as reversibility, unobtrusiveness and minimum alteration; Phase 4—execution of the works with appropriate quality assurance and skilled personnel; Phase 5—monitoring, as an assessment of the effects related with the changes. The current study focus exclusively on Phase 2 mentioned above, viz. in the aspects of numerical analysis applied to the existing building infrastructure in the northern areas of Pakistan that are vulnerable to strong ground motions. The major part of existing building infrastructure have not yet suffered the consequence of a moderate or strong earthquake, meaning that there is no proficiency on their actual performance under seismic actions. The state of structural safeguarding of the existing buildings is not well known, being the key purpose for working the present vulnerability study.

The concept of sustainability is frequently used in the areas of construction economics or green building all together, the structural typology adopted being less taken into account, also including the earthquake resistance [6]. Reinforced concrete (RC) structures, given their dominance, are generally considered as a reference for sustainable structural design. It has been noticed in some major seismic events that masonry structures suffered more seriously than did concrete structures such as in 1990 Manjil [7], 1971 San Fernando and 1994 Northridge earthquakes [8]. If an accurate nonlinear study of masonry buildings be carried out, the security of the building is augmented and the cost can habitually be abridged. Therefore, the understanding of their seismic response is essential so as to assess the seismic behavior of masonry structures. As a consequence of high computational costs mandatory to execute dynamic time-history analysis, the non-linear pushover analysis, is a very smart method to approximate the seismic performance of assemblies due to its ease and effectiveness [9–11]. Non-linear pushover analysis is usually used to approximate the actual displacements and forces established in the structural components due to ground motion. Thus, non-linear pushover analysis is adopted in order to determine the capacity curve of URM and CM structures in this presented paper.

After major seismic hazards happened in the past three decades (1987 Whittier, 1989 Loma Prieta, 1990 Manjil, 2003 Bam and 2005 Kashmir) the requirement for using more sophisticated techniques for assessment of seismic demand on masonry structures became obvious. Currently the finite element method is the broadest and one of the most prevailing tools for the assessment of masonry structures. Two practices exist for modeling the response of masonry buildings including: macro and micro-level modeling. A method for investigation of unreinforced masonry structures that is used mostly is the macro-modeling of masonry as a composite material. This method is more practical due to the lessen time and memory necessities in addition to a user-friendly mesh generation. In the numerical method, numerous authors have suggested different models for calculating the behavior of masonry buildings under different loading and boundary conditions.

Milani [12] in 2006, offered a micro-level model for the homogenization limit analysis of in-plane loaded masonry. The precision of the model has been evaluated by significant comparisons both with kinematic methodologies. A 3D kinematic FE limit analysis of full masonry structures subjected to lateral loading to determine the response was done by Milani [13] in 2007. Six dimensional linear homogenized surfaces for masonry were gotten and executed in a finite element program [12,14].

Pasticier [15] in 2008, performed the seismic analyses of masonry structures. To model the two unreinforced stone-masonry walls in the Catania Project, SAP2000 was adopted. By comparing the numerical calculations acquired through nonlinear analysis and results of the Basilicata research group have established the ability of the suggested model in giving close estimates of base shear forces. Though, different outcomes from SAP2000 and SAM code which was established by the University of Pavia, were projected. Cecchi and Milani [14] in 2008, proposed the macroscopic failure planes of two-wythe masonry prepared in English bond pattern by adopting Reissner–Mindlin kinematic limit analysis method. The norms were that flow rule is related both for the materials models and a finite subclass of likely deformation ways.

Park [16] in 2009, calculated the seismic fragility of a low rise unreinforced masonry structure. The used structural modeling technique uses a simple, composite nonlinear spring. The unreinforced masonry panel was divided into different areas or sections that are characterized by nonlinear springs.

Rota [17] in 2010, worked on offering a new analytical methodology for deriving the fragility curves of masonry structures. The procedure depends on nonlinear probabilistic analyses of structure models. To generate input measures from the probability density functions, Monte Carlo simulations were adopted.

Milani [18] in 2011, offered a homogenized model for the nonlinear and limit analysis of masonry piers/panels in-plane loaded. The nonlinearity was assumed to be concentrated on unit-unit interface and the mortar joints were reduced to interface. Nonlinear analyses were performed over laboratory test data. Akhaveissy [19] in 2011 suggested a new closed form solution to define the shear strength of unreinforced masonry panels. Prophesied results showed less error percentage than the ATC and FEMA-307 [20]. To model the mechanical behavior of mortar in masonry piers, the numerical execution of a new suggested interface model was adopted. The hypothetical background was derived from plasticity theory. Also Akhaveissy and Desai [21] in 2011 offered an integrated model to describe the behavior of masonry assemblies based on Disturbed State Concept (DSC) with improved hierarchical single yield surface (HISS) plasticity. The assessments of suggested model with outcomes from test data exhibited proper accuracy.

Milani [22] in 2012 estimated the seismic performance of the Maniace Fort in Siracusa, Italy under lateral load using 3D Finite Element discretization method. The structural response was studied in detail by using different methodologies. A comparison with the model demonstrating the fort in its original arrangement was also provided. Akhaveissy [23] in 2012 offered a nonlinear FE technique with eight-nodded isoparametric quadrilateral elements to calculate the response of masonry structures in-plane loaded. The disturbed state concept (DSC) with improved hierarchical single yield surface (HISS) plasticity was adopted to describe the constitutive relation of masonry in both tension and compression. Akhaveissy [24] also in 2012 offered a simple proficient procedure based on diagonal strength of URM piers to evaluate capacity curve of URM structures. The Von Mises principle was adopted to simulate the response of the units. Many masonry assemblies, with low- and high-rise masonry structures, were studied using the offered procedure.

Akhaveissy and Milani [25] in 2013 suggested a simple, 2D macroscopic FE model for the lateral load response of actual scale masonry buildings in-plane loaded. To describe the behavior of masonry the so called disturbed state concept (DSC) with improved hierarchical single yield surface (HISS) plasticity model with allied flow rules were adopted.

As demonstrated, numerous models have been established in the previous two decades with macro and micro level modeling [26]. The results discussed associated to macro-modeling procedures presented significant differences among different approaches of macro-modeling in comparison with actual test data (Pasticier [15]; Akhaveissy [24]). Furthermore, the use of those methods to calculate the response of masonry structures and further extending its use in common professional practice is not entirely extended. Hence, the proposal of new models that can be applied to full scale real life structures with simplifying and reducing the modeling and computational cost and allowing for suitable precision of the entire procedure of nonlinear analysis of real life masonry structures is of great significance. Nevertheless, the widespread practice of these models needs first a clear demonstration of its precision and consistency. Therefore, in this investigation, a macro model is discussed using 2D nonlinear shell elements in a finite element framework. Nevertheless, optimizing building performance in general (economy, resilience, sustainability, etc.) calls for a holistic methodology to sustainability, which must necessarily take into account the structural typology [27]. This study reports the seismic assessment of real life masonry structures, also concentrating on the economy features in construction. A calibration of current macro-model approach for URM and CM structures is performed by comparing with experimental results. The performed validation permits to spread importantly the application area of the considered modeling approach [28].

Later, by the application of this method to the next generation performance-based earthquake assessment of archetypal house buildings in Pakistan, the URM and CM structures are assessed and compared with the RC structures. Referring to archetypal single-family dwellings in Northern parts of Pakistan and their typical design, the validated assessment method is used to assess the seismic design and evaluation of URM and CM structures considering that these structures look to be viable substitutes to RC structures.

An experimental program with displacement control lateral loading on URM and CM structures, is acquired to assess the calculations by the monotonic analysis adopting the macro-element models. Numerous experimental studies on masonry structures have been done worldwide. The behavior of shear walls have been investigated by Epperson and Abrams [29], Abrams and Shah [30], Magenes and Calvi [31], Anthoine et al. [32],Manzouri et al. [33], Tomazevic et al. [34], Craig et al. [35], Franklin et al. [36] and Calderini et al. [37]. Similarly, Simsir et al. [38] worked out the out-of-plane behavior of walls. Nevertheless, these studies did not explained the response of local construction materials and practices specific to masonry construction in northern Pakistan. The masonry building chosen was a single-story structure, representing the interior room of a typical housing system. Fig. 1 presents the comprehensive illustration of the structure. Opening dimensions and size of piers for in-plane walls were adopted in accordance with the wall density ratio and pier aspect ratios of characteristic brick masonry structure in Pakistan. An opening was also introduced in an out-of-plane wall as well to investigate the flange effect. The tested URM and CM buildings, offered in Shahzada [39] and Asfandyar [40] respectively, are comprised of solid clay brick masonry structures, with a dimensions of 3048 mm × 3658 mm (10 × 12 feet) and the height of 3353 mm (11 feet).

The walls were 9″ (229 mm) thick with English bond configuration. In the case of unreinforced masonry (URM), the lintel beams 9 in. wide and 6 in. deep were provided over all the openings, reinforced with four ½ inches bars in longitudinal direction and 3/8 in. stirrups at 6 in. center-to-center. However, in the case of confined masonry, reinforced concrete confining elements were provided around all the openings as well as at all the corners of the building. The thickness of these confining elements was equal to wall thickness with the width of 6 in.. The reinforcement details of these elements were similar to that described for the lintel beam. Toothing was also provided as a mechanical bond between the confining elements and the masonry. The primary purpose of these elements is to enhance the lateral load capacity and displacement ductility to the masonry piers/walls. In both the building typologies, a 6 in. thick slab, reinforced in both the directions with ½ inch bars at 9 in. center-to-center was provided.

A 13.5 in. wall was given all over the slab on center line of structural walls, to represent the vertical load effect from the contiguous building parts. To account for the dead load representing the roof treatment, a 10 in. thick layer of sand was applied on the top of slab. The structure was erected on a 7.5 in. thick concrete footing fixed to the strong floor, reinforced in both the directions with ½ inch bars at 6 in. at centre. Both the full scale masonry structures are presented in Fig. 2.Material testing was also performed with these large scale tests to determine various properties of masonry. The description of these properties is given in Table 1.

## 2. Macro-element models for masonry

Masonry structures possess explicit and different bond typologies. To address that, various modeling methodologies have been practiced. Generally, for the academic purposes the masonry modeling has been practiced at two diverse scales, called as micro- and macro-element methodologies [41]. When using micro level or meso level models [42–44], masonry inherent orthotropic behavior is considered with definite nonlinear constitutive models for mortar, brick–mortar interface and masonry units, which are modeled distinctly. Whereas this permits an accurate estimate of the masonry behavior under diverse loading conditions, it is unfeasible in the analysis of real life masonry structures due to the extreme computational work required,

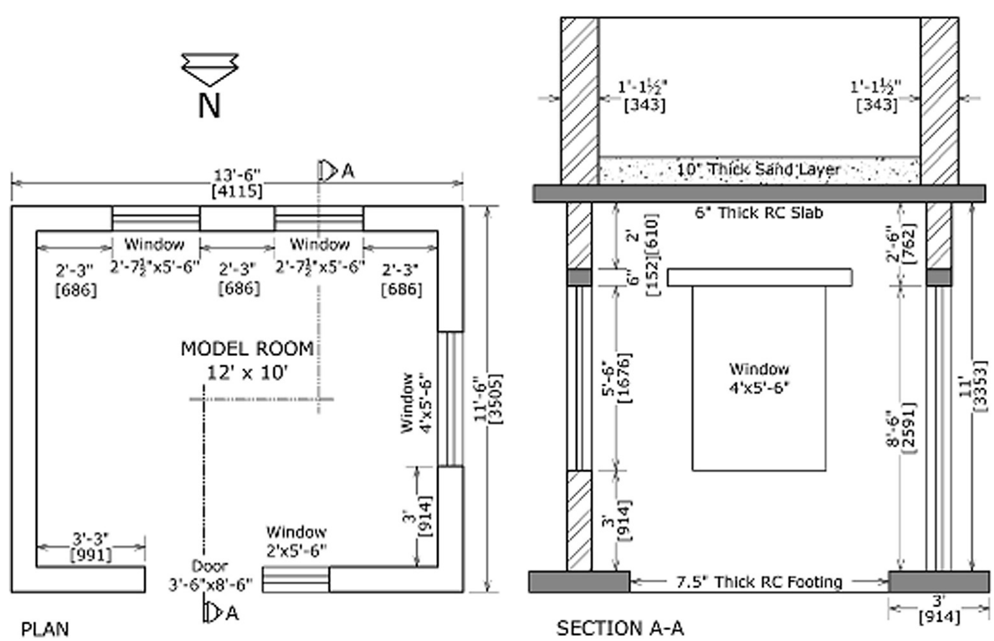



**Fig. 1.** Structure Details (dimensions in brackets are in mm) [39].

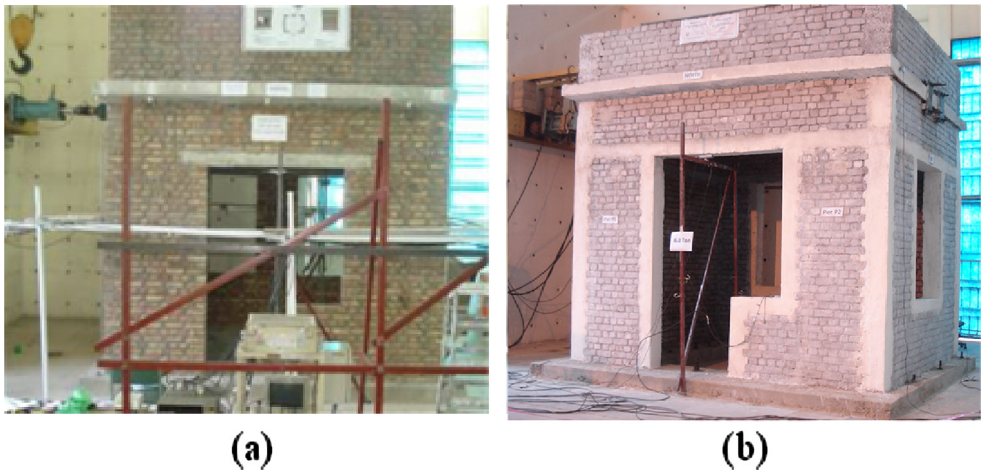



**Fig. 2.** Full scale masonry structure (a) unreinforced masonry [39], (b) confined masonry [40].

**Table 1**
Material Properties [39].

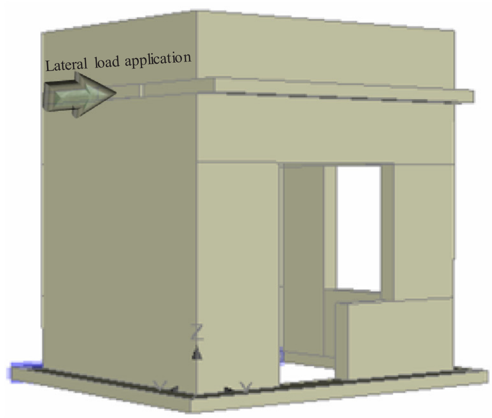
| Symbols | Description | Results | COV (%) |
|---|---|---|---|
| $f_b$ | Masonry unit compressive strength, psi (MPa) | 1803 (12.43) | 26.7 |
| $f_{bt}$ | Modulus of Rupture, psi (MPa) | 479 (3.30) | 20.7 |
| $f_m{'}_o$ | Compressive strength of mortar, psi (MPa) | 733 (5.05) | 26.6 |
| $f_m{'}$ | Masonry compressive strength, psi (MPa) | 438 (2.61) | 27.2 |
| $E_m$ | Elastic modulus of masonry, ksi (MPa) | 178 (1228) | 37 |
| $f_{tu}$ | Masonry diagonal tensile strength, psi (MPa) | 7.3 (0.05) | 23.3 |
| C | Cohesion, psi (MPa) | 3.22 (0.022) | 0.92 (SD) |
| μ | Coefficient of friction | 0.21 | 0.92 (SD) |

particularly when three-dimensional (3D) modeling is used to represent masonry elements with a multifaceted bond or the interaction of the in-plane with out-of-plane behavior [44]. Field applications of these hypothetical methodologies can be seen e.g. in Ramos et al. [45], Lourenco et al. [46], Carol et al. [47], Brocks et al. [48], Seguarado et al. [49], Alfano et al. [50] and Macorini et al. [44], but they stay limited to somewhat small group of specialists. The idea of adopting structural element models for masonry structures, labelled by “macro-element modeling” was presented in the 1970s by Tomazevic [51] and adopted to do seismic evaluation. In macro level approaches, masonry is normally idealized as a homogeneous material, and explicit damage or plasticity-based formulations are applied to account for material non-linearity [42,52]. This idea is discussed next, with the smooth execution of constitutive models and formulating the structural equilibrium. The assumed structural element discretization essentially decreases the number of degrees-of-freedom as compared to the customary modeling methodologies, permitting for more resource-efficient and time-effective calculations, so that practitioners find them attractive and applicable. In the next section, the existing models are briefly discussed and corroborated, for unreinforced masonry (URM) and confined masonry (CM).

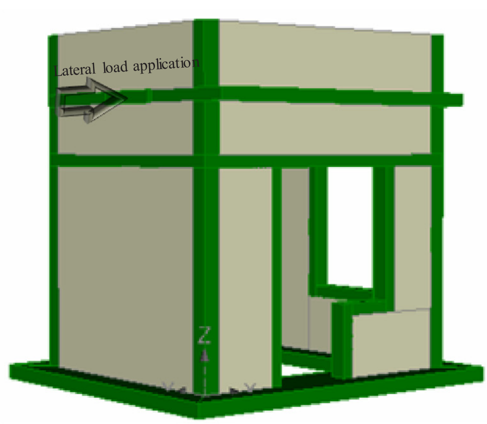
### 2.1. Models for unreinforced masonry (URM)

In recent times, numerous comprehensible computer codes on the basis of macro-elements have been introduced for evaluating the potential safety of masonry buildings. Rinaldin et al. [53] and Lourenco [54] presented a modelling strategy to describe the nonlinear behavior of unreinforced masonry (URM) elements subjected to in-plane cyclic loading, which can be used for seismic assessment of masonry structures. Magenes developed the ANDILWall [55], Lagomasino developed the TreMuri [56] and Calio developed the 3DMacro [57] software codes, and delivered the elementary portrayal of the macro-element design and accumulation adopted in these techniques. For further details, the readers are referred to Marques [58]. The main limitation of these tools is that they idealize a three dimensional structure into a two dimensional structure, thus compromising the representation of the actual behavior of a real three dimensional structure. Therefore a commercially available non-linear finite element software ATENA [59] was adopted for modeling of URM structures in this study, which allows for the assessment of both global and local seismic response of buildings. The masonry wall was modeled using Ahmad Shell element proposed by Ahmad et al. [60]. It considers both plane and bending structural stiffness. The element presents quadratic geometry and displacement estimation and therefore, the shape of the element can be non-planar. It is possible to consider the structural curvatures. The main advantage of this element is that it is smoothly connectible to true 3D elements (available in Atena). The Ahmad element associates to group of shell element formulation that is established on 3D elements’ concept. Nevertheless, it applies some restrictions and assumptions, so that the actual 3D element is converted into 2D space only. It not only saves computational time but also avoids some formulation complexities relating to 3D elements. The in-plane integration of the element is carried out in typical way by Gauss integration method, whilst in the third dimension (i.e. along the thickness of the element) the integration can be performed in closed (analytical) form. However, to consider the nonlinearity of constitutive model, the layer concept is used instead. Hence, in the third-dimension simple quadrilateral integration is adopted. All the solid walls were modeled as a single shell element. However, masonry wall with opening is idealized by recognizing two main structural components, namely the spandrels and the piers. The piers are the major vertically resisting elements while the spandrels combine the reaction of two adjacent piers. This concept was developed from the monitoring of typical damage in past earthquakes [28]. The in-plane stiffness of the diaphragm (i.e. horizontal floor) is also an important parameter while describing the role in coupling the response of the different masonry walls (i.e. in-plane and out-of-plane). And the assumption in modeling the diaphragm as rigid, semi-rigid or flexible will completely change the seismic response of the masonry structure [61–63]. These few comments emphasize the consideration that must be paid by researchers while analyzing the seismic response of existing masonry buildings through global numerical models. While in some cases simple models could be used, however a global model can better evaluate the seismic behavior of the building if proper modelling of walls and floors is done.

The macro-element model applied in ATENA [59] enables the two major failure modes governing the behavior of masonry walls to be replicated. Regarding flexural behavior, rocking and toe crushing




(a)




(b)

**Fig. 3.** Macro-element modeling details of (a) unreinforced masonry and (b) confined masonry structures.

mechanisms are considered, whereas shear sliding, and diagonal cracking are considered for shear failure. The real three dimensional structure was modeled with the same dimensions and boundary conditions in accordance with the actual tested structure in the laboratory. The seismic evaluation is done by performance based methods, i.e. pushover (nonlinear static) analysis, as per modern design codes [2–4].

The experimental arrangements are presented in Fig. 2a, where an actuator was positioned at the slab level for the application of lateral load to the building. Story drift ratio is calculated as a ratio of lateral displacement to the story height at the lateral load level. The stiffness degradation happened with increasing displacement and stiffness became almost zero at story drift of 0.20%. The structure achieved its maximum lateral strength at a story drift of 0.23% after which strength degradation initiated at a slow rate. The lateral strength was degraded by 5% at a story drift of 0.41%.

The tested URM building was modeled in the finite element software mentioned above, as presented in Fig. 3a. For the URM structure the capacity curves acquired are compared in Fig. 4 with the experimental results. Fig. 4 presents the comparison of the numerical and experimental response curves. In the numerical model the first indication of damage starts right after the elastic threshold (i.e. after drift level of 0.030%), where a small horizontal branch develops which indicates the rocking behavior of the model. Since the actual structure was composed of small brick units joined together by mortar, however in the numerical model the entire wall/pier was modeled as a single assembly, thus difference in response was obvious. Nevertheless, on the whole, the numerical model offers adequate estimation of the experimental response curve, regarding the maximum lateral load and initial stiffness. Furthermore, the model replicates the stiffness degradation in the subsequent part of the experimental behavior. Beyond the good approximation perceived in the global behavior of the assembly, it is interesting to examine the load progress on the different walls, in addition to the progression of damage by increasing the applied load.

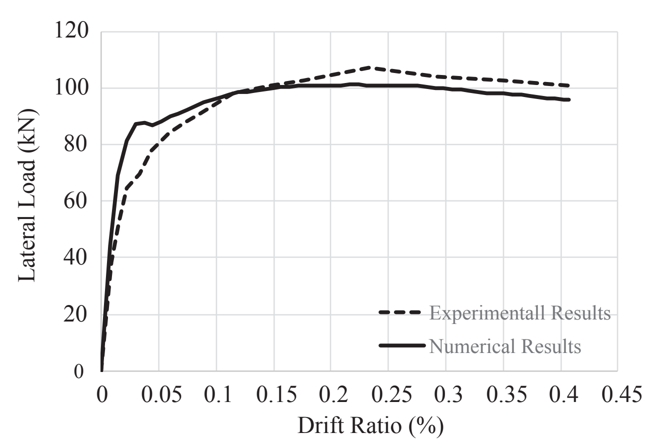



**Fig. 4.** Experimental vs Numerical results for unreinforced brick masonry structure.

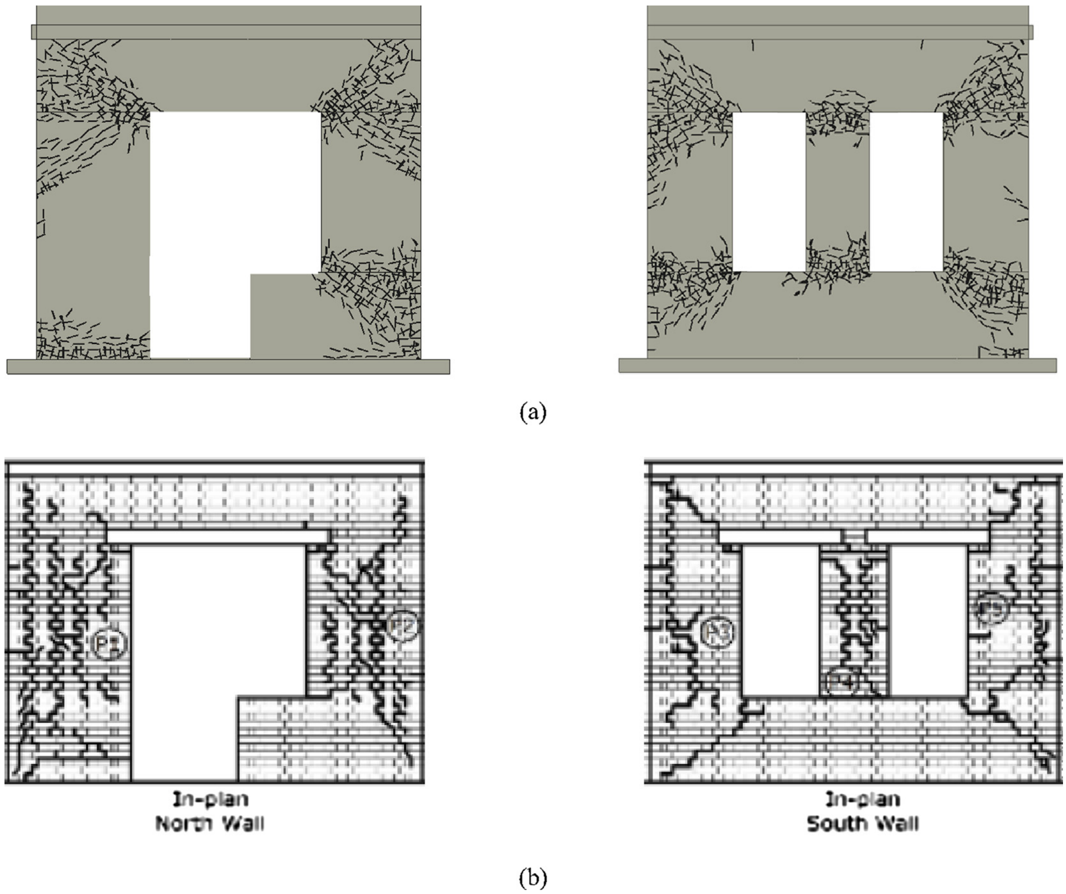



**Fig. 5.** Comparison of (a) Numerical and (b) Experimental Damage Patterns of URM.

Fig. 5 presents the comparison of damage pattern of numerical model and the experimentally tested model. It can be seen that the damage in the in-plane walls is more concentrated in the piers, in both the experimental as well as numerical models. However, the central pier of the in-plane south wall seems to develop a diagonal cracking failure in the experimental test, while in the numerical model, rocking failure is observed. This can be due to the initial calibration error and can be removed with further research on such topic and enhancing the existing models as well. It can also be seen in Fig. 5 that in the numerical model, 45 degree shear cracks appeared in the in-plane north wall on both the top corners of the opening, while no such cracks are observed in the experimental results. The reason could be associated with the provision of lintel beam in the experimental test setup, however no such beam was provided in the numerical model thus resulting in different damage pattern and crack orientation than the actual one. This observation clearly emphasize the role of lintel beam in taking the load from the spandrels above the lintel and transfers the load to the piers on both sides of the opening. Similarly, in the south wall of the numerical model, the damage is more concentrated at spandrel level above the openings. However in the actual tested model, due to the presence of lintel beams, the damage is transferred from the spandrel to the piers underneath.

This behavior is also stated by Cappi [64], where the cracking in the walls was usually by shear damage and was commenced in the in-plane piers. Therefore, the Atena model by considering clearly both diagonal shear and flexural damages for the piers possibly offers a more precise estimate. The deformed shape and damage mechanism is presented in Fig. 5. A broader classification for the wall prophesied response is illustrated in Marques [65]. The available test results relating the wall damage state and deformed shape are limited, however as stated by Cappi [64] the wall behavior combines wall and frame-type response configurations. Because of the wall-type response, the overturning moment produces compression in the right and tension in the left piers, however the spandrels are principally subjected to shear.

### 2.2. Models for confined masonry (CM)

Confined Masonry is a specific instance of masonry constructions, although it offers some resemblance with reinforced concrete (RC) constructions because of the existence of a surrounding frame. Confined masonry is described by forming the RC components simply after the masonry works, which offers a better connection of the confining elements with the masonry piers because of the mechanical bond effects due to toothing, shrinkage of the concrete and the point that the gravity load is resisted primarily by the masonry walls. The interface performance of the confining elements with the masonry is a definite feature that must be taken into account in the behavior of confined masonry structures subjected to seismic loading. Some models have been employed for confined masonry buildings established on a wide-column method, e.g. [66,67], allowing for the contact behavior among the confining elements and the masonry piers implicitly in the shear behavior of the pier.

Micro-modeling approaches can also be adopted, explicitly based on the finite element technique, to model plainly the unit-mortar interface, e.g. Calderini [68]. Otherwise, a distinct element methodology is also relevant, for example the one idealized by Caliò [69] initially for unreinforced masonry. In this study, a structure tested under quasi-static conditions by Asfandyar [40] at Earthquake Engineering Center of University of Engineering and Technology Peshawar, Pakistan, is considered to estimate the finite element method employed in the Atena software [59] for confined masonry constructions. The structure, along with the plan is presented in Fig. 1, resembles to an inside room of a

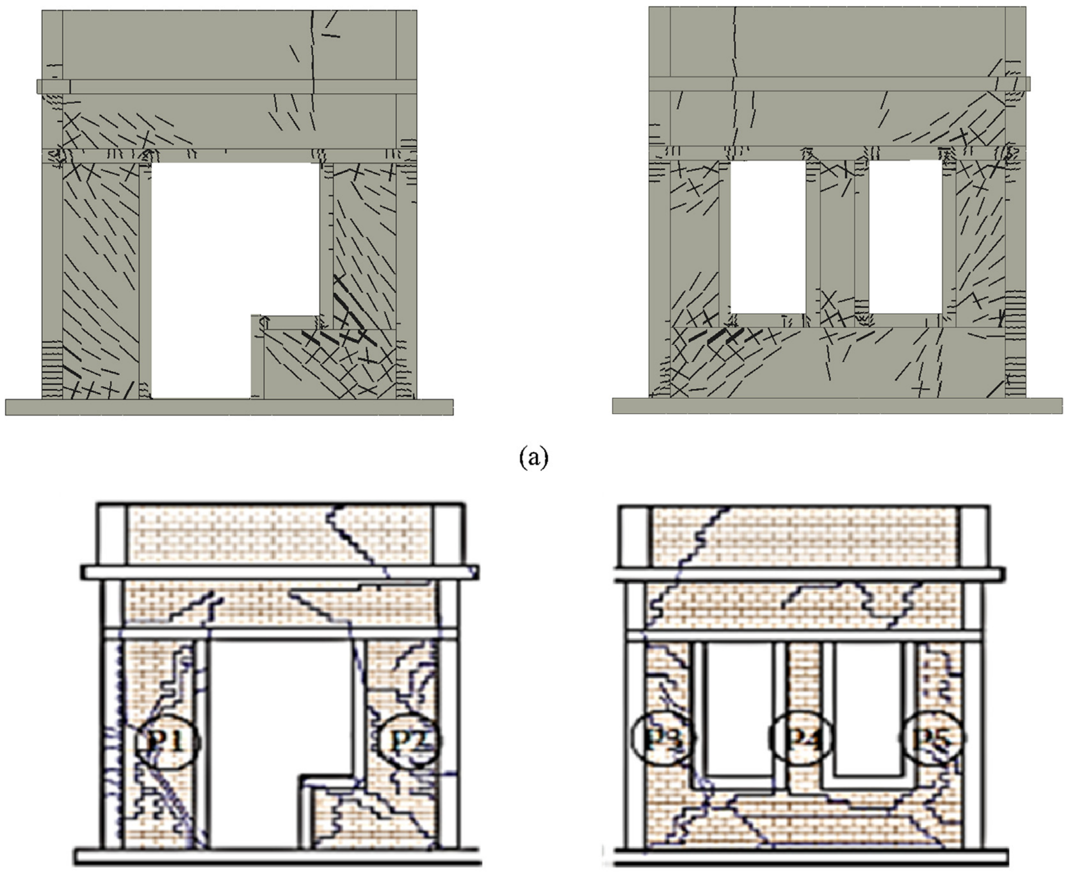



**Fig. 6.** Comparison of (a) Numerical and (b) Experimental Damage Patterns of CM.

typical house in Pakistan. The building is constructed with clay bricks laid in English bond pattern, confined on all sides by RC elements.

The experimental arrangements are illustrated in Fig. 2b, where two actuators (acting together to push in the same direction) are positioned at the slab level for the application of lateral load to the building. Displacement-controlled environment was adopted for the application of the loading. Regarding the structure damage progress, the assembly is stated to have acted in elastic way up to a lateral drift ratio of 0.03%, whereas cracking initiated at 0.093% drift ratio. The building attained its peak resistance at 0.65% story drift, then the strength degradation happened at a slow speed. The continuing deprivation of the building started past the story drift of 0.65% of maximum resistance. However, the testing was stopped after the lateral load strength reduced to 89% of the peak loading corresponding to 0.73% story drift.

The structure was modeled in Atena as shown in Fig. 3b. In the analysis, the expected damage happens generally by diagonally cracking of the confined masonry panels, and also by the formation of plastic hinges (flexural) in the confining elements. A comparison of the experimental and numerical damage progression is presented in Fig. 6 for both the in-plane walls.

The lateral load capacity curves for the confined masonry structure are compared in Fig. 7 against the tested one. The numerical curve is in good agreement with the tested envelope, replicating the initial stiffness and maximum lateral strength very accurately. However, the post peak strength degradation as found in the tested structure couldn't be captured by the numerical model. This could be because of the fact that the damage was more pronounced in the mortar joints in the experimentally tested structure, while in the macro-model approach masonry is modeled as homogeneous material thus compromising the ability to predict the local failure which happened at the mortar joints.

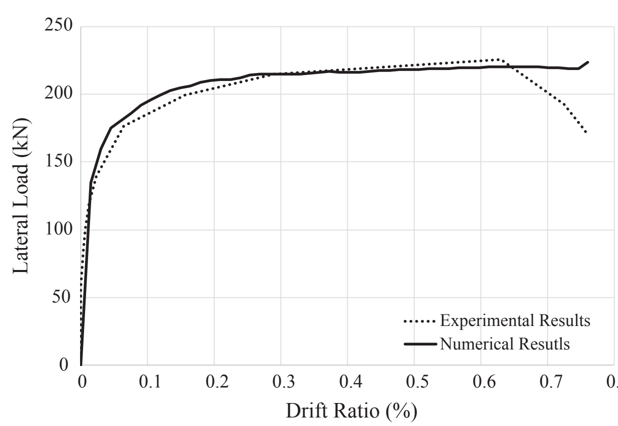



**Fig. 7.** Experimental vs Numerical curve for confined brick masonry structure.

## 3. Comparison of different structural solutions for a dwelling

Building structures up to three storys comprise the major proportion of the building stock in Pakistan, in terms of present as well as recently built structures. The forfeiture of masonry in terms of a structural solution, because of an ungrounded opinion of its deficient capacity to resist seismic actions and also to the increasing popularity of RC constructions, resulted in a strong decline of the practice of masonry in new constructions. Conversely, a huge development happened in the masonry engineering, explicitly with the instigation of superior masonry methods concerning functional and mechanical aspects. In the educational field significant efforts have been made to progress satisfactory

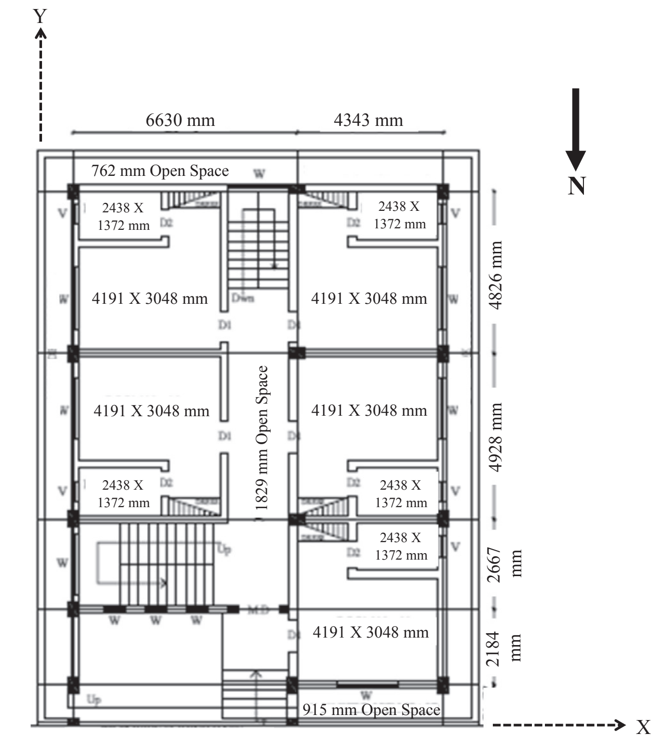



**Fig. 8.** Plan details (Ground floor) of the case study buildings, showing X (transverse) and Y (longitudinal) directions.

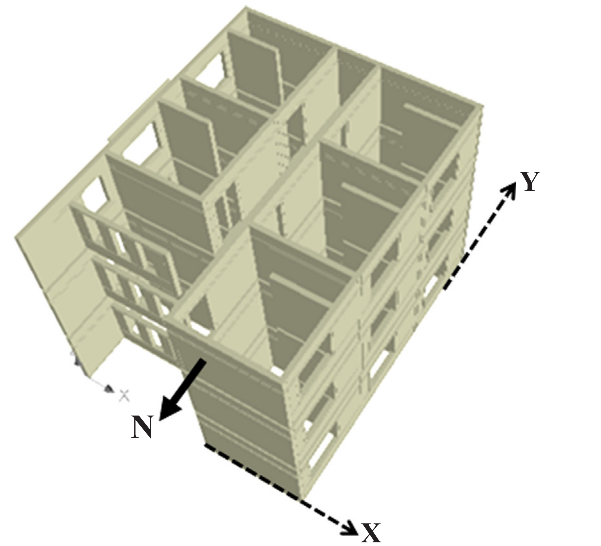



**Fig. 9.** Three dimensional computational model of a typical three story building structure.

tools to represent the essential nonlinear response of masonry assemblies when subjected to lateral loadings. An initial comparison of such tools was performed by Marques and Lourenço [67] discussing a simple structural configuration. In the current study, the comparison is extended, regarding an actual and more complex construction, discussed in the next section.

### 3.1. The case study

The adopted single, double and triple story dwellings are representative of characteristic housing in the northern Pakistan, with kitchen, living room, bed room and drawing room in the ground story and bedrooms in the first and second story (Fig. 8). A typical three story computational model of the building is presented in Fig. 9. The structure was actually designed as RC construction, and hence also modeled in this study for the sake of comparison purposes so that the two adopted masonry alternatives can be evaluated and compared to the standard RC option. The advantages and disadvantages of both the opted solutions can thus be better understood. In the succeeding, URM and CM solutions are offered and compared with the RC construction. The structure is supposed to be constructed on type D soil (stiff soil) with prevalence to seismic hazards with magnitude lower than 7.5.

### 3.2. Unreinforced masonry solution

Unreinforced Masonry, allowing for a moderate structural solution that permits an efficient energy enclosure using no thermal bridges, is the primary implemented option. Here, a solid clay brick masonry structure is adopted.

The system uses a solid brick with dimensions $0.230 \times 0.115 \times 0.076$ m and corresponding pieces (half-brick, king closer etc.) permitting a plan geometry with a wall thickness of 0.23 m. The plans for the suggested solution are offered in Fig. 8, which show the structural details of the proposed building.

Regular reinforced concrete slab was taken for all the floors construction with a uniform thickness of 0.15 m. The slabs modeled are analogous to the original RC structure slabs. The material properties considered for masonry were according to Shahzada [39], and are presented in Table 1. All the three typologies i.e. single, double and triple story structures were modeled in the same manner as presented in Section 2.1. The base of the model was fixed and lateral load was applied at the roof level for all the typologies. All the three models are presented in Fig. 10.

All the three models were subjected to pushover loading in both the principal directions. Regarding the +X and +Y direction analysis as per the requirements of FEMA P-58 [70], the damage pattern for the buildings is presented in Figs. 11–13 (for +X direction only) corresponding to ultimate roof displacement levels. For +X and concerning the facade, an analogous style is observed in the ground story (for all the three typologies) with shear failure of the middle pier and rocking of the end piers, and with displacement concentration at ground story. In all three building typologies, the flexural failure in the out of plane walls and diagonal shear failure in the in-plane walls is noticed at all the story levels.

A similar damage mechanism and considerable shear damage are detected for other wall configurations. In +Y loading direction, the models are in agreement while predicting rocking and shear failures for slender and squat panels respectively. The lateral load response for all the three structures in both the principal directions is presented in Fig. 14.

### 3.3. Confined masonry solution

Confined masonry presumed as a transitional typology between unreinforced masonry and reinforced concrete constructions, is now taken into account by means of the prescriptive guidelines from Eurocodes [71].

In the case of vertical confining elements (tie-columns), as per EC6 [71], a minimum longitudinal reinforcement ratio of 0.8% must be provided, with a minimum bar size of 12 mm diameter. The shear reinforcement comprises of 9.5 mm diameter stirrups at 0.20 m center to center. The structural plans for the CM solution are presented in Fig. 15.

The confined masonry assemblage was modeled in the Atena software [59], with the computational model presented in Fig. 3. The major change relative to the unreinforced masonry model is the addition of the RC confining elements (bond beams and tie columns), which are modeled as continuum elements having a relatively strong interface (as

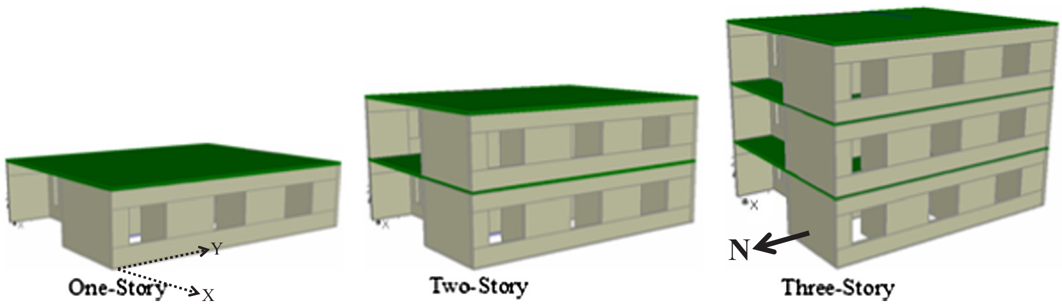



**Fig. 10.** Unreinforced masonry solution for all three story levels (showing X (transverse) and Y (longitudinal) directions).

compared to unreinforced masonry assembly) with the adjacent macro-masonry elements. The interface between confining elements and masonry was modeled using a gap element based on coulomb-friction criteria which imply the frictional in tangential direction and "hard contact" in the normal direction with quadratic interpolation. The masonry piers/walls were modeled as shell element to reduce the mesh complexity. The pushover analysis is then performed in both the principal directions in the way similar to that of unreinforced masonry, as discussed previously.

The damage mechanism for the confined masonry models for single, double and triple story options is presented in Figs. 16–18 for the + X direction, corresponding to ultimate roof displacement levels. Regarding the analyses, cracking in all the structures initiated at 1 mm roof displacement, while numerous masonry panels experienced tensile cracking and similarly diagonal shear cracking occurred. At a roof displacement of 4 mm cracks spreads extensively, to all panels of the structure. The results presented in Fig. 18 regarding the damage pattern of three-story structure matches very well with the results presented by Alcocer [72], where the out-of-plane walls were damaged due to flexural cracking while diagonal shear cracking was the dominant failure mode in the in-plane walls. Penetration of diagonal cracking to tie-columns ends was recorded in that study and can also be seen in this case as well.

After that the damage progresses with increment in the roof displacement because of the further development of plastic hinges in the confining elements (i.e. tie-columns). The resistance curves are presented in Fig. 19 for all the three options of single, double and triple story structures in both the + X and + Y directions. A comparison between the unreinforced and confined masonry solutions is presented in Figs. 20–22. The lateral load capacity in both the directions is seen to increase considerably for confined masonry solution as compared to unreinforced masonry. For single story structure, the lateral load capacity increases more than 300% for confined masonry solution, however for double and triple story structures, an increase of 70% and 50% has been observed, respectively for confined masonry construction. A sharp decline of capacity is observed for URM structures after the peak, because of improper connection between the orthogonal walls of unreinforced masonry structure, especially in the case of double and triple story structures.

In the case of confined masonry construction, the existence of confining elements incorporates a restraint for the masonry panel, which persuades a damage mechanism different from that of unreinforced masonry. Referring to damaged structures shown in Fig. 18, it is perceived that for the URM structure the cracking is initiated by tensile stresses in pier corners and with succeeding rocking of the piers, however for the CM structure the damage initiated due to diagonal/shear cracking which extends through the wall diagonals persuading flexural hinges in the confining elements. For a lateral deformation of 08 mm, the URM structure is utterly damaged, whereas the CM structure presents extensive cracking but damage is controlled.

### 3.4. Comparative analysis

An assessment based on the analyses has been performed in much depth about the lateral load capacity of URM and CM structures. Specifically, the behavior aspects to use in displacement-based safety confirmations is discussed. About financial features, an assessment is done for the construction prices of the URM and CM solutions with the typical RC structure. Figs. 20–22 presents the comparison of behavior of URM and CM structures along both the longitudinal and transverse directions. It can be seen very clearly that for all the three story levels, the lateral load response of confined masonry structure is far better than the unreinforced masonry both in terms of lateral strength and ultimate displacement. However, this difference in lateral load response is greater for single story construction in both the longitudinal and transversal directions than the other two story arrangements. This is due to the fact that adding more stories to URM structure causes the increase of axial load on masonry walls which mainly suppressed the tensile field in a material inherently weak in tension.

Nevertheless, the displacement capacity of URM is very low as compared to CM, especially in the case of two and three story structures where a sudden drop down is noticed after the peak lateral strength, thus indicating a relatively brittle behavior due to addition of axial load on the structure in terms of number of stories above. In all types of structures i.e. one, two and three story, the enhancement in displacement capacity appears to be more definite than the enhancement in lateral load capacity. Accordingly, the energy absorption capacity is also improved considerably. It can be concluded that the inclusion of confining elements do not improve the lateral load capacity of masonry structure considerably, but they mainly modify the ductility of structure, the stress distribution and damage mechanism in masonry structure in a positive way. The rise in energy absorption and displacement capacity describes the effectiveness of confined masonry structures against extensive damage that did not collapse in previous major

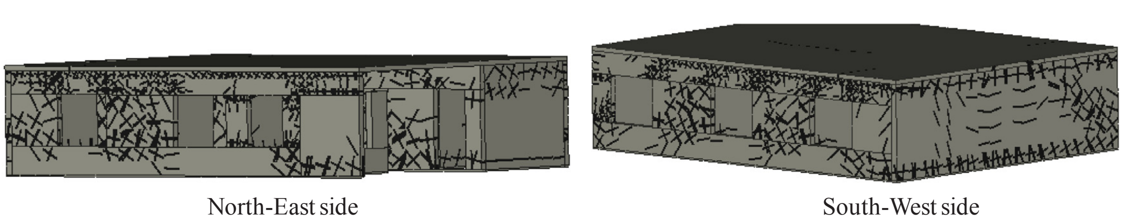
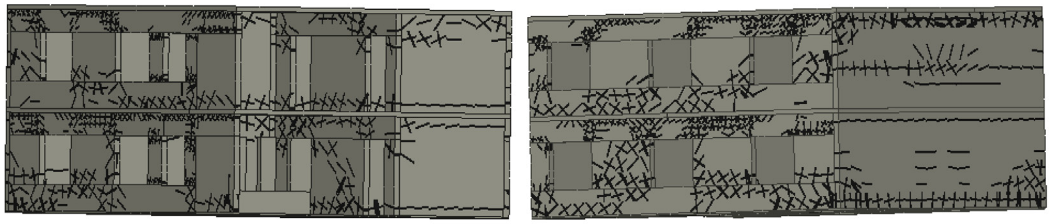



**Fig. 11.** Damage Mechanism of single story unreinforced masonry structure.



North-East side South-West side

**Fig. 12.** Damage Mechanism of double story unreinforced masonry structure.

earthquakes whereas unreinforced masonry structures were destroyed completely.

## 4. Cost analysis

To adopt a structural solution regarding the cost-effectiveness, the expenses related to the construction of the structure with URM and CM typologies are now approximated for a single story structure. By assuming 2017 costs in the Pakistani construction market (market rate system 2017 based on composite schedule of rates, CSR 2017) [73].

### *4.1. Building's description*

The type of masonry is same for URM and CM structures to make the comparison more accurate in terms of economy. The geometrical characteristics of buildings are the following: to keep the same opening size (doors, windows and ventilators etc.) the wall size in case of CM was reduced to add the confining elements around each opening, thus reducing the brick work and increasing the reinforced concrete work (i.e. formwork installation, reinforcement fixing, casting of concrete etc.). The reinforced concrete class (i.e. steel grade, concrete type etc.) is taken same for all RC elements (e.g. foundation, slab, confining elements etc.). Since all the walls act as load bearing element in masonry construction, thus proper foundation must be provided under all the longitudinal and transverse walls. Therefore wall footing of reinforced concrete with 305 × 305 mm cross-section were considered under all the walls in masonry construction. The cross sectional area of tie-columns were taken as 228 × 228 mm in CM case and the cross-sectional area of 228 × 152 mm was taken for bond beams in case of CM and lintel beams in case of URM construction. The floor and the slab above the ground area are 160.2 sq. m, the walls are 293.65 sq. m (URM) and 252.25 sq. m (CM), and the opening area (windows, doors and ventilators etc.) is 42.56 sq. m for both URM and CM.

### *4.2. Results and discussions*

The wall density in both the directions was determined according to Building Code of Pakistan-Special Provision 2007 [74] and UBC-97 [75] codes considering the seismic zone and group of masonry units.

As can be noticed in Table 2, the wall density is greater than 4.5% in transverse direction and greater that 5.5% in longitudinal direction for both types of construction, thus fulfilling the minimum requirements in both the directions.

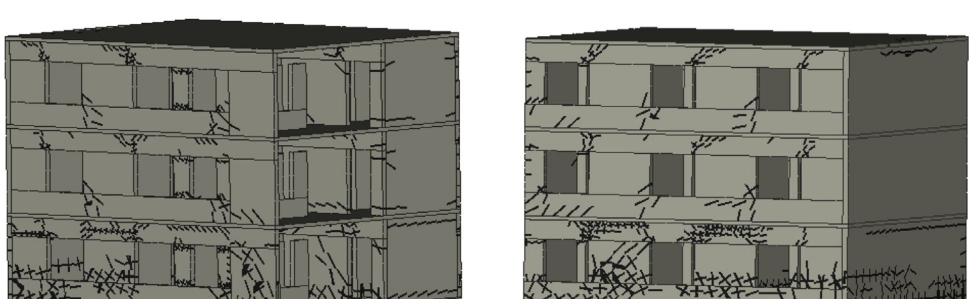
In case of URM, there are no tie-columns in the building thus the brick work volume increases in comparison with the CM where tie columns are provided at each corner, wall junction and around each opening (i.e. window, door and ventilator) by keeping the size of opening similar to that of URM. Also in URM, the lintel beams are provided at top of the opening, however in CM the bond beams are provided both at top and bottom of opening to make a better bond of tie-columns. In this way the brick work in CM reduces as compare to URM, however the reinforced concrete work increases.

The prices of the different structural solutions are illustrated in Fig. 23. The values presented in the figure are as a ratio (%) of the complete cost of the single story RC solution. Regarding the masonry structural solutions, there is substantial reduction in the cost of foundation, because of small sections as the stresses are relatively smaller and distributed all over the walls, and also cost reduction in the RC construction. Note that there is not much difference in cost of masonry work for the CM and URM since the brick work in URM is greater than CM, this is because of cutting of units and provision of toothing with tie-column in the CM that results in more brick work in this type of construction. As a whole, the URM and CM structural solutions permit a total cost saving of, correspondingly, 28% and 20% as compared to the RC solution. This improves to a quicker construction method and improved long service performance as a result of enhanced resilience and reduced damage and residual deformation.

It should be mentioned that in this analysis of costs, the cost of workmanship is not taken into account and the supplementary costs that might have led to a relatively higher difference reported to the finishing cost.

## 5. Conclusions

This study proposes a contribution about the design and development of cost-effective structures in seismic areas. For this reason, the tools offered for the seismic assessment of unreinforced masonry (URM) and confined masonry (CM) structures are presented and corroborated



North-East side South-West side

**Fig. 13.** Damage Mechanism of triple story unreinforced masonry structure.

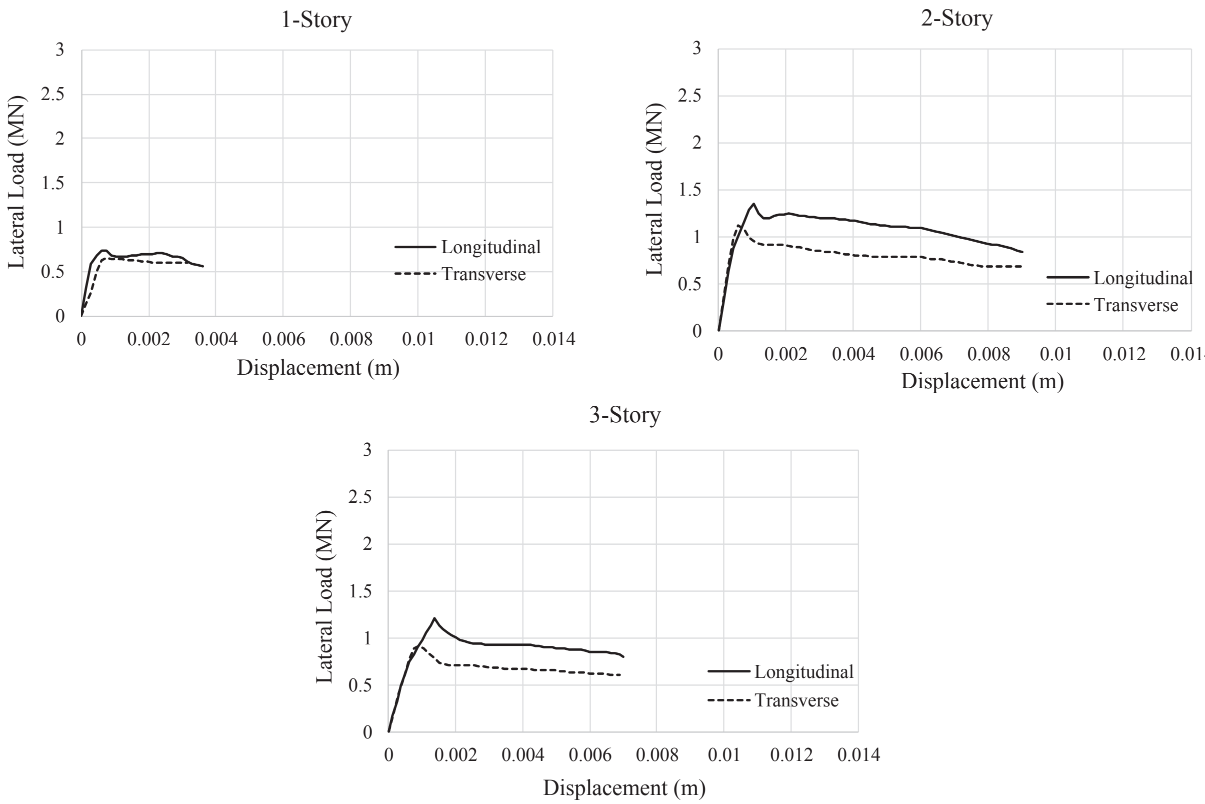



**Fig. 14.** Pushover curves for unreinforced masonry structures in X (transverse) and Y (longitudinal) directions.

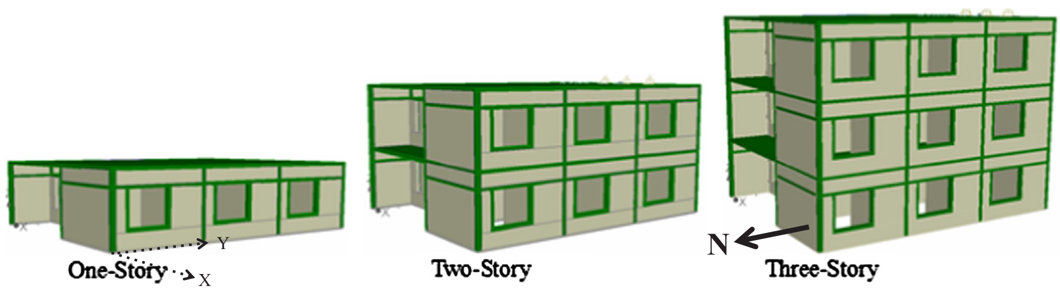



**Fig. 15.** Confined masonry solution for all three story levels (showing X (transverse) and Y (longitudinal) directions).

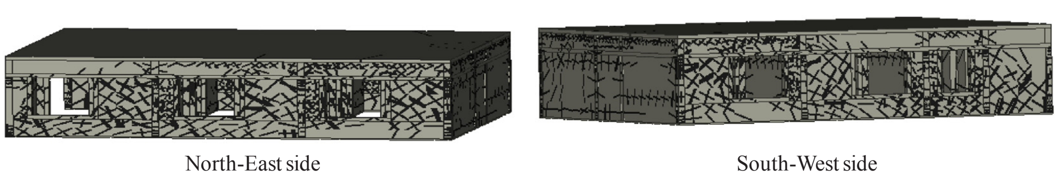



**Fig. 16.** Damage Mechanism of single story confined masonry structure.

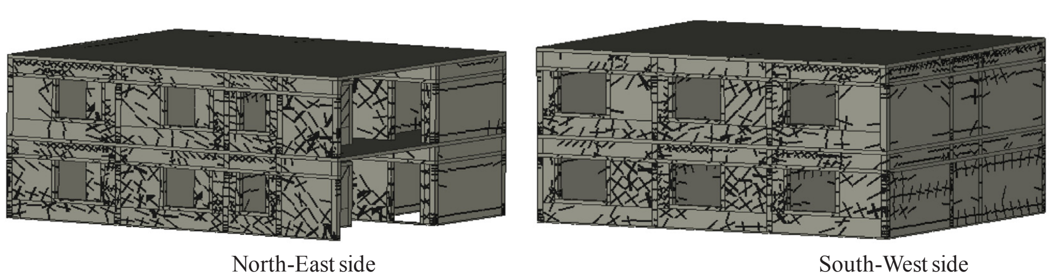



**Fig. 17.** Damage Mechanism of double story confined masonry structure.

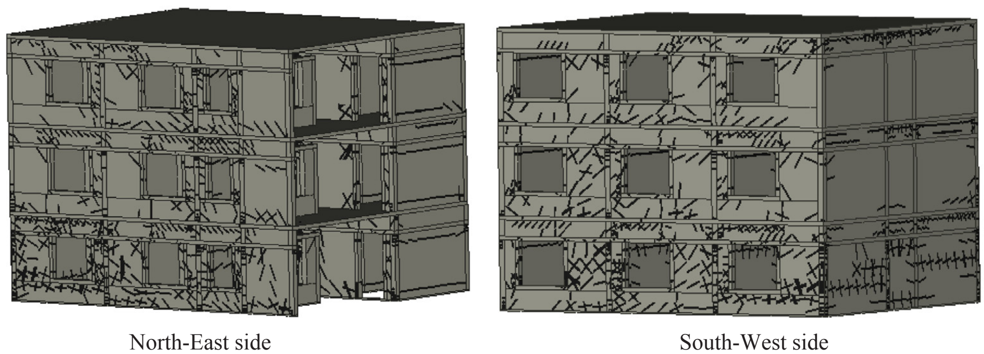



**Fig. 18.** Damage Mechanism of triple story confined masonry structure.

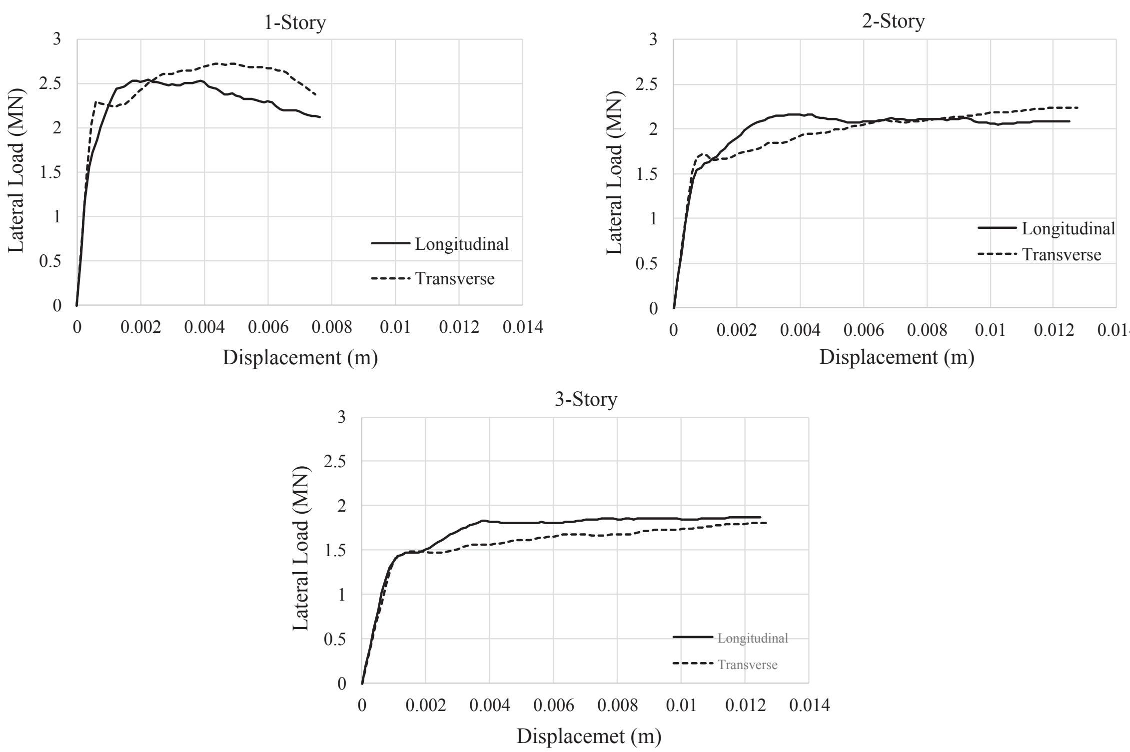



**Fig. 19.** Pushover curves for confined masonry structures in X (transverse) and Y (longitudinal) directions.

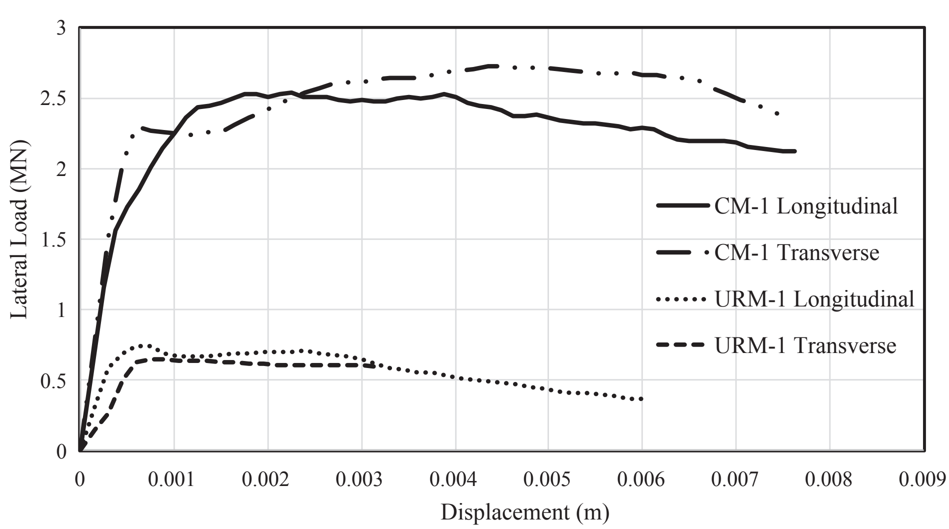



**Fig. 20.** Comparison of single story structures for confined and unreinforced masonry solutions.

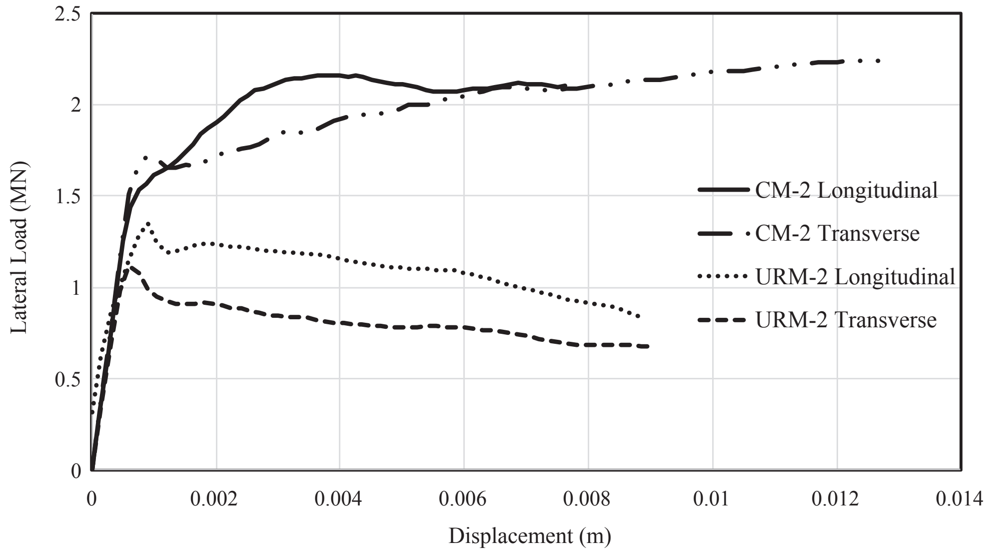



**Fig. 21.** Comparison of double story structures for confined and unreinforced masonry solutions.

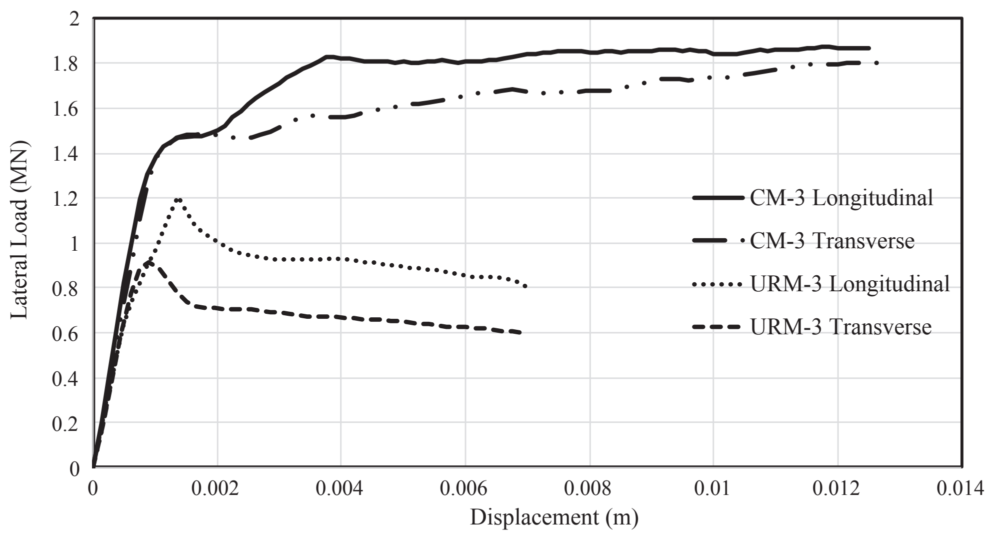



**Fig. 22.** Comparison of triple story structures for confined and unreinforced masonry solutions.

**Table 2**
Characteristics of Structural Masonry Walls.

| Typology | URM Building | | CM Building | |
|---|---|---|---|---|
| Building Orientation | Longitudinal Direction | Transversal Direction | Longitudinal Direction | Transversal Direction |
| $A_{wall}$ | 147.05 | 146.60 | 127.81 | 124.44 |
| $A_{Opening}$ | 32.52 | 10.04 | 32.52 | 10.04 |
| Wall Density (%) | 6.0% | 4.8% | 6.0% | 4.8% |

against experimental indication. In general, the assessment tool presented, provided a reasonable estimate of the capacity curve from the lateral load analysis, specifically initial stiffness, ultimate strength and ductility, hence being precise enough to be adopted in performance-based assessment. Additionally, the software permit simulating the configurations of actual masonry buildings. Regarding a genuine case of a single-story, double-story and triple-story dwellings, simulations are performed to evaluate its earthquake-resistant performance and an assessment of the cost of construction is made for masonry solutions in comparison with the typical RC frame. Using the archetypal dwelling in the northern Pakistan as case study, the usual masonry solutions permitted confirming ductility range up to nominal lateral displacement levels, an average of 8 mm and 12 mm for URM and CM solutions, correspondingly. Thus it can be concluded that by confining the masonry piers/walls, the ductility capacity and hence the energy dissipation capacity of the structure increases considerably and increases the resilience against the seismic activities. Similarly the lateral load capacity of the confined masonry structures also increases as compared to unreinforced masonry by 300%, 70% and 50% for one, two and three story structures, respectively. Moreover, these structures offers a reduction in cost of the structure correspondingly of 28% and 20% when comparing with the reference RC structure. The replication of the actual dynamic response of a masonry structure is a very multifaceted job. To abridge, the validation of the current macro-element models with the experimental results and also its use to a characteristic structure permits to get an approach to the pushover behavior of the considered structures, which is supposed to offer a descriptive indication of the seismic response, specifically about the application of seismic codes.

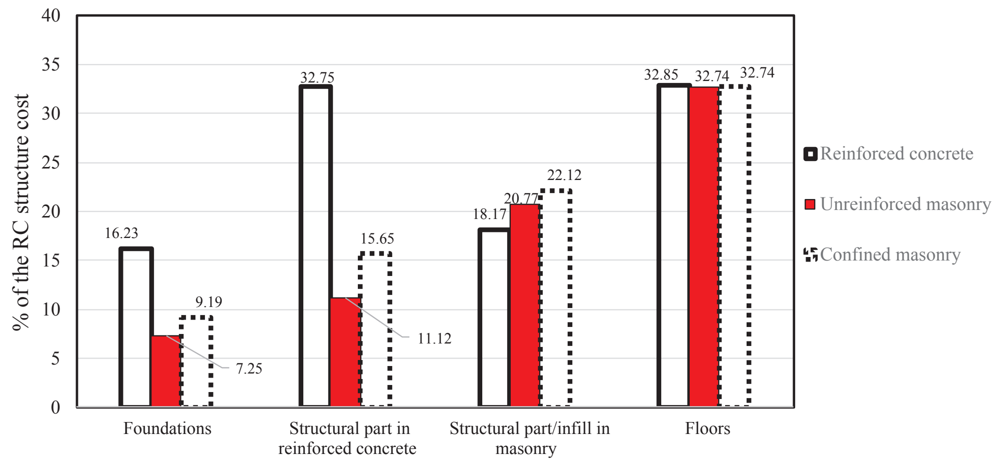



**Fig. 23.** Summary of construction costs for different structures.

## Declaration of Competing Interest

The authors declared that there is no conflict of interest.

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## Appendix A. Supplementary material

Supplementary data to this article can be found online at https://doi.org/10.1016/j.engstruct.2019.109612.

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