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A basis for defining archetypes for non-engineered low-rise unreinforced brick masonry residential buildings of Pakistan

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ABSTRACT

The salient characteristics of low-rise unreinforced brick masonry residential buildings of Northern Pakistan are presented, aiming to provide a basis for defining archetypes suitable for seismic vulnerability assessment in this area. Firstly, an overview of the typical geometrical characteristics, the mechanical properties of the commonly used materials, the main structural features, and observed damage patterns of such buildings is given. Available test data on a set of wallets and a single room consistent with the configuration and construction practices in Northern Pakistan are selected. The wall strength reference values and the effect of construction issues that influence the boundary conditions, such as the wall-to-wall and wall-to-slab connections, are studied, too. The consistency between the failure mechanisms activated by the experimental tests and those predicted by analytical strength criteria available in the literature is shown; this is used as a tool to define the most plausible values of mechanical parameters to be adopted as a reference in seismic safety building evaluation. These results constitute the preliminary but necessary step to address structural models for building and class vulnerability assessment of, arguably, one of the most dominant and vulnerable building types in Pakistan, and by extension, of a large part of South Asia.

Keywords: unreinforced brick masonry, Pakistan, strength criteria, non-engineered structures

INTRODUCTION

The built-up area that is lost on average per year due to earthquakes concerns mostly the non-engineered structures of the developing countries of Asia, based on the results of the Global Seismic Risk model of the Global Earthquake Model (GEM) Foundation (Silva et al., 2019). In developing countries, the high seismic vulnerability of vernacular structures is combined with high social vulnerability due to low-risk awareness, high occupancy rate, large household size, and the predominant young age group. In Kijewski-Correa et al. (2010), the empowerment model is presented for the sustainable post-earthquake reconstruction in Haiti. Porst et al. (2017) propose confined masonry as a solution for seismically resilient low-cost housing in India and other developing countries; moreover, in this work, a seismic design procedure for confined masonry that can be easily captured in an Excel spreadsheet is proposed. In Bothara et al. (2018), many recommendations are provided about the earthquake risk reduction efforts in Nepal, and in Bothara & Sharpe (2009), an insight is given into many “paradoxes that have led us to tolerate unsafe buildings”.

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In the present work, Pakistan is used as a case-study country. In 1935 the severe Quetta earthquake occurred in the area of Balochistan in Pakistan. However, after 1935, apart from the municipality of Quetta city (Naseer et al., 2010), there was no significant change neither in the building codes nor in the construction process. This is true especially for the residential structures in urban and even more in rural areas. In the Unreinforced Brick Masonry (URBM) structures, only Reinforced Concrete (RC) lintel or sill parts might be provided, which are placed above or below the windows but not perimetrically. In 2007, the severe Kashmir earthquake happened. Afterwards, a new building code (BCP-2007) towards confined masonry was adopted, and half a million buildings were reconstructed within 3-5 years.

For the case at hand, the non-engineered low-rise (1, 2-story) URBM residential dwellings with a heavy RC slab were selected to be the studied building type mainly for three reasons. First of all, these URBM buildings are vulnerable to seismic action, but by learning from past earthquakes (Javed, 2008), they have the potential for better performance than stone masonry or adobe structures. Secondly, the inherent strength, the good insulation, the low cost, the need for less knowledge of engineering compared to RC frames, the construction by locals, and the local availability of used materials are some of the main reasons that lead the URBM construction to be is the most dominant building type (PCO, 1998; Mahmood & Ingham, 2011, Maqsood & Schwarz, 2011) in urban areas, big cities in Pakistan (mainly Northern Pakistan, i.e., Muzaffarabad, Islamabad, Balangot, Abbottabad) except Karachi (Ali, 2006). In Karachi, RC frames with concrete blocks or brick infill walls are the most populous building type. In rural areas, URM adobe and mud houses are the most dominant building classes, with URMs in clay mortar being the second one. However, there is a trend of replacing the adobe and mud houses with URMs with cement sand mortar or cement sand khaka mortar. URMs in rural areas are characterized by lower strength as for the brick class and mortar quality compared with that of urban areas; moreover, the roof may be with iron sheets or Bamboo. Thirdly, considering the urbanization and high birth rate, they are projected to dominate the existing building stock of the studied region.

Some studies that are based on URBM components or URBM structures with the most recurring configuration and commonly used construction materials in Pakistan are already available in the literature, such as a) the experimental investigations of a set of masonry constituents, assemblages as well as walls (Javed et al., 2012; Ali et al., 2012) and a full-scale single room that replicates an internal room within a building (Shahzada et al., 2012); b) the dynamic field test with underground explosions of a single room (Ali & Naeem, 2007); c) the numerical seismic-resistant analysis of a structure that was initially designed for an RC construction (Ahmed et al., 2019); d) the use of an example building in Northern Pakistan as the basis for a methodology to assess the lateral strength (Javed, 2009); d) a simplified approach for earthquake loss estimation that adopted a case study URBMs representative for Mansehra of Pakistan (Ahmad et al., 2010). The present study assembles all this essential information about the recurring characteristics of the selected building type and then aims to define the reference values of the mechanical parameters to be adopted for their seismic assessment. Starting from experimental evidence, the work aims to set a basis that can be used to pass from a case-study URBM to a set of building cases representative for the studied building class. These building cases, that carefully identify the most dominant features with significant impact on the structural and seismic response, are typically called archetypes or index buildings (Reitherman & Cobeen, 2003; D’Ayala et al., 2013; Kazantzi et al., 2014). The selected URM is also a common building type in urban areas of other South-Asian countries such as India, Bangladesh, etc. Thus, the presented work is expected to be useful for creating structural models for building and class vulnerability assessment for, arguably, the most vulnerable buildings of Pakistan, and by extension, of a large part of South Asia.

Once the selected class of interest was defined, our work is based on two pillars, supported by the expert opinion about the local practices and construction trends in Pakistan. The first is the collection of the essential information and then the identification of the most recurring features. This data collection is mainly categorized, following D’Ayala et al. (2013), into three subsets: 1) building type configuration and dimension such as the number of stories, plan and front views, 2) mechanical characteristics, 3) geometry characteristics and structural detailing such as the thickness of walls, lintels, and boundary conditions. A supplementary subset is added about the observed damage patterns. The second pillar focuses on defining the reference values of mechanical parameters for the analytical strength criteria to be adopted in the safety verification of URM panels and the structural models.

A BRIEF OVERVIEW OF WALL STRENGTH CRITERIA AND EXPERIMENTAL DATA SETS

From the observation of seismic damage in masonry walls, as well as from experimental tests, it has been highlighted that masonry piers subjected to in-plane loading can be characterized by two typical behaviors: a flexural behavior that involves the rocking and crushing failure mode; a shear behavior that produces sliding or diagonal cracking failure mode. The occurrence of different failure modes depends on several parameters, such as the geometry of the pier, the boundary conditions, the axial load, the mechanical characteristics of masonry constituents (mortar, blocks, and interfaces), the masonry geometrical features (aspect ratio of the block and cross-section pattern). Many experimental or numerical investigations have attempted to analyze the influence of these parameters on the failure mode of masonry piers (e.g., Calderini et al., 2009; Albanesi&Morandi, 2021; Magenes&Calvi, 1997).

The simplified evaluation approaches of the shear strength of URM walls can be roughly divided into two categories: the ones that consider masonry as an equivalent isotropic material (Turnšek & Cacovic, 1971; Turnšek & Sheppard, 1980), and those that describe masonry as a composite material (Mann & Müller, 1982). The two approaches mainly differ in the choice of reference stress (e.g., shear, normal or principal stress) and a reference section where failure is initiated, providing different predictions of the shear strength. Their reliability depends on the degree of anisotropy of the type of masonry examined. In particular, two main parameters determine these different behaviors: the chaoticity of the masonry pattern, and the ratio between the strength/stiffness parameters of mortar and blocks.

The formulation, according to Turnšek and Cacovic (1971), originated from experimental tests on unreinforced masonry piers with double-fixed boundary conditions. It considers that a diagonal crack occurs when the principal stress at the center of the wall reaches the critical value, namely the tensile strength of masonry. The maximum shear strength V_t is provided by the following formulation:

$$V_t = l\,t\,\frac{f_{tb}}{b}\sqrt{1+\sigma_0/f_{td}} \tag{1}$$

where f_{td} is the diagonal masonry tensile strength (usually determined by the diagonal compression test), l is the panel length, t is the wall thickness, σ_0 is the normal tension referred to the total area of the section (i.e. equal to $P/(lt)$, P being the axial force). f_{td} is then usually related to the masonry shear strength τ_0 (i.e. $f_{td} = 1.5\,\tau_0$), as adopted for example in the Italian Structural Code (MIT, 2019) where reference values for τ_0 are also proposed for different masonry types. The parameter b, equal to the in-plane slenderness h/l of the wall (h being the height of the panel), is limited as proposed by Benedetti & Tomaževič (1984) between 1.0 and 1.5.

On the other hand, the formulation proposed by Mann and Müller (1982) is based on the experimental study of shear-compression tests on masonry walls. Two main failure mechanisms were identified: the failure in the mortar joints and that in the masonry units. The limit shear strength V_t is provided by the following formulation:

$$V_t = l\,\frac{t}{b}\left(\frac{f_{v0d}}{1+\mu\Phi} + \frac{\mu}{1+\mu\Phi}\sigma_0\right) \le V_{t,lim} \tag{2}$$

$$V_{t,lim} = \frac{l\,t}{b}\frac{f_{btd}}{2.3}\sqrt{1+\sigma_0/f_{btd}} \tag{3}$$

where f_{vod} is the masonry shear strength in the absence of normal tension; μ is the friction coefficient that according to MIT (2019), can be assumed equal to 0.577; Φ is the interlocking parameter defined as the ratio between the height and overlap lengths of the masonry units. $V_{t,\text{lim}}$ is intended as a limit value, function of the tensile strength of the units f_{btd}. Note that for the calculation of the interlocking coefficient Φ, especially in bond patterns different from the typical and most utilized "running or stretcher bond", for example in the case of the so-called "English bond" walls built with an alternate stretcher and header courses, the minimum overlapping length between units of two adjacent courses should be assumed as the mean between the two cases (Magenes & Calvi, 1997).

Finally, the flexural behavior is governed by the ultimate bending moment (M_u) of URM walls that, for the sake of simplicity, can be evaluated by assuming a proper stress distribution for the masonry in compression and neglecting the tensile strength of the bed-joints; in general, the classic equivalent rectangular compression diagram "stress block" at the compressed toe with an ultimate compressive strength equal to $0.85f_d$ is assumed (where f_d is the compressive masonry strength). Hence, M_u can be computed as

$$M_u = \frac{l^2 t\, \sigma_0\, t}{2}(1 - \frac{\sigma_0}{0.85\, f_d}) \quad (4)$$

and the corresponding maximum shear strength under flexural behavior becomes

$$V_f = \frac{M_u}{\xi h} \quad (5)$$

where: h is the pier height; ξ is a factor of boundary condition equal to 2 or 1 in case of fixed-fixed or fixed-pinned ends, respectively. It is worth noting that Equations (1) to (4) are also adopted by the draft of EC8-3 (CEN, 2020), the Italian Structural Code (MIT, 2019), and the New Zealand code (NZSEE, 2017).

As reference data sets, two experimental investigations are examined, namely that of Javed et al. (2012) and that of Shahzada et al. (2012). Both are representative of the URM structures in the Northern areas of Pakistan and therefore exemplify the most recurring configurations (geometry, construction patterns, etc.) and commonly used construction materials. In the experimental work of Javed et al. (2012), in-plane quasi-static cyclic tests were performed on URM shear walls constructed in stone dust mortar. This stone dust mortar was used as a partial replacement for sand and is called locally “khaka”, from “khak”, which means clay. There was a double fixed condition in the experimental setup and a limit of 1360mm in the length of each wall due to the capacity of the actuator. Shahzada et al. (2012), performed a full-scale experiment of one story URM structure that replicates an internal room within a building. The 9-inch-thick brick (i.e., 0.025m) walls were made with burnt clay bricks in an English bond pattern, i.e., with alternate courses of stretchers and headers, the rigid reinforced concrete slab 0.15m thickness, and the reinforced concrete lintels above the openings are the main components of this single room.

RECURRING FEATURES OF URM WITH RC-SLAB IN PAKISTAN

The considered buildings are non-engineered structures, but they are primarily skilled construction, i.e., English masonry bond and toothing are applied. However, they are constructed to support only gravity loads without any seismic design. Therefore, they are good in compression but show less resistance to the shear forces resulting from earthquake forces. Their weakness of tensile strength is also combined with the high weight (becoming also larger due to the RC slab) that leads to larger seismic forces. Additionally, structural deficiencies are related to mortar properties and the class of brick. Furthermore, the row housing without any seismic gap in urban areas may add a potential pounding effect.

The occupancy type is usually residential, although there are also some commercial structures. The layout and the main characteristics that affect the structural response are quite similar all over the big cities of Pakistan apart from the mountain areas. The most relevant information about the building type configuration concerns the number of stories, corresponding heights, and plan and front view details. The typical number of stories is one or two, while less commonly, three. The net story and ground floor height are in the range of 2.0 to 3.0 m, in the case of 130mm thick RC slab, and 2.5 to 3.5 m, in the case of 150 mm thick RC slab (Ahmad et al.,2010). The main plan shape is rectangular solid with dimensions of 5, 7, and 10 Marlas. The Marla is a traditional unit of area that is used in India, Pakistan, and Bangladesh. The 5, 7, and 10 Marlas are equal to about 126, 176, and 253 m^2, respectively. Ali (2006) reports that, in the urban areas of northeast (NE) Pakistan, the most prevailing dimensions are 9.15 m×18.30 m (5 Marla) and 18.30 m×27.40 m (10 Marla). Their plan aspect ratio is 1:1.5 (length parallel to the road: length perpendicular to the road, Mahmood & Ingham, 2011).

Fig.1a depicts one/two-story isolated URM in Pakistan denoted with the capital letters A and C, respectively. The cases A1(C1), A2(C2), and A3(C3) represent small, medium, and large residential buildings, respectively; the case A4(C4) refers to a residential housing type in a small town or village, while case A5 (C5) to a village residential property (Mahmood & Ingham, 2011). The plan and the front view of a single room are shown in Fig 1b; the room configuration replicates an internal room within a representative URBM building in Northern Pakistan (Shahzada et al., 2012).

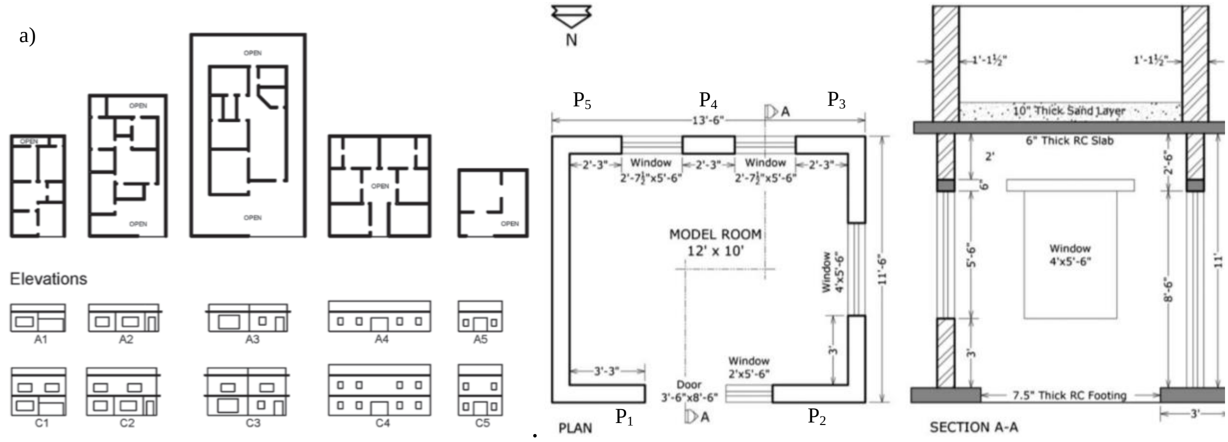

Figure 1. a) floor and front view plans of 5 typologies (Mahmood & Ingham, 2011) b) *The plan and front view of the single room representative for URBM building in Northern Pakistan (Shahzada et al., 2012)*

Table 1 shows the mechanical properties of masonry as derived from the available literature on this building type. The masonry compressive and diagonal tensile strength are in general very low due to the weak bond between mortar and brick because of the high Initial Rate of Absorption (IRA). The IRA of the units is greater than 30grams/min/30in^2, indicating that it must be wetted well before being employed in the construction of masonry works. The major three types of mortar are a cement-sand mortar, a cement-sand-khaka mortar, and a cement-khaka mortar (Ali et al., 2012). In Ali et al. (2012), relatively higher shear strength and stiffness were reported due to the use of the khaka in mortar, in addition to the better workability and economy in construction work.

Table 1. *Main mechanical properties of masonry and its constituents*

	(Javed et al., 2012)	*(Shahzada et al., 2012)*	*(Ali,2006)*	*(Ahmad et al., 2010)*
Masonry compressive strength f_d (MPa)	4.53	2.61	2-5	4.83
Masonry diagonal strength f_{td} (MPa)	0.24	0.05	0.2-1.6	0.15
Masonry coefficient of friction of μ	0.59	0.21	0.2-0.8	0.62
Compressive strength of brick units(MPa)	22.5	12.4	16.91	17.24
Mortar cubes 28 days compressive strength (MPa)	5.05	5.05	2.5-25	6.2

Photos of the construction process and details of walls appear in Fig. 2. These buildings primarily have 230 mm thick solid load-bearing double burnt-brick walls without cavity, an RC slab 130–150 mm thick, and RC lintels only above the openings (Fig. 1b). There are no vertical posts (either wooden or concrete). The presence of a ring beam depends on the construction period. The size of the brick masonry unit varies from 215 to 230 mm in length, 100 to 110 mm in width, and 60 to 70 mm in thickness. In Pakistan, the brick units are produced in three major classes: well-burnt bricks with uniform size and sharp edges, underburnt bricks with a high percentage of cavities having a non-uniform size, and overburnt bricks with irregular shape and size. The first class of bricks is mainly used in buildings in urban areas. The solid fired clay brick, with a small cavity on one face called frog, is prepared manually in a kiln. The second class is mainly used in rural areas, while the third class is used for filling depressions and raising ground-floor levels (Ahmad et al., 2010). A shallow foundation is adopted with reinforced concrete strip footing. The cast-in-place beamless reinforced concrete floor and roof are constructed as rigid RC slabs, simply laid on top of the perimeter walls. The one and two-story masonry residentials were/are built mainly by builders who might ask for advice from an engineer about the region with confinement and the RC slab.

***Figure 2**. a) Construction process and structural detailing b) lintel and wall bonding view, and c)poor quality of RC slab construction, bricks are added in RC roof-slab* (Ali, 2006)

Concerning the in-plane response of walls, the main damage patterns are the diagonal shear failure and the flexure failure of the piers; moreover, out-of-plane effects and failure of building corners (Fig. 3d) have been observed. In Fig. 3, some of the typical damage patterns are shown. The main causes of failure are the heavy weight of the roof, the slenderness of the walls, the absence of vertical posts and ring beam, the relatively large openings in some cases (Fig 3a), and the quality of the mortar (Maqsood &Schwarz, 2008). A typical problem of the considered type of masonry is the out-of-plane failure of the walls with the sudden falling down of the heavy roof (Fig. 3b). The large inertial forces attracted by the heavy RC roof, the slenderness of the walls (high height and inadequate wall thickness), combined with the insufficient connection to the roof, are not able to withstand the inertial forces. That is a mechanism with very brittle failure. Additionally, since there are no vertical posts, the cracks run throughout the wall (Fig. 3a) without any resistance (Maqsood & Schwarz, 2008).

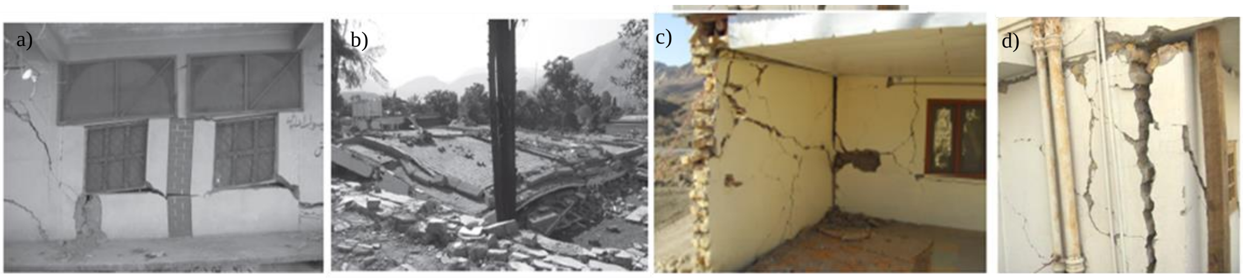

***Figure 3**. Typical damage patterns of URBM structures in Pakistan, a) shear cracks (Maqsood & Schwarz,2008), b) heavy roof (Maqsood & Schwarz, 2008), c) collapse of walls and bond failure (Maqsood & Schwarz, 2010), and d) separation of orthogonal walls (Javed, 2009)*

WALL STRENGTH BASED ON STRENGTH CRITERIA AND EXPERIMENTAL DATA

The analytical strength criteria summarized in the previous section are applied to the tested set (Javed et al., 2012) of four wall series (W1, W2, W3, W4); for each series, three wallets were tested. In Javed (2009), experimental results are provided for one tested wallet of the W1 series, three tested wallets W2a, W2b and W2c for the W2 series, and two tested wallets for the W3 series i.e. W3a and W3b. The fourth series of wallets, due to the difference in the behavior compared to all other wallets, is not considered further since it was also excluded in the original research. Table 2 summarizes the geometry of panels (different aspect ratios, i.e., height/length) and the pre-compression levels of the three wall series. The mechanical parameters are instead shown in the first column of Table 1. The considered pre-compression levels aimed to simulate the vertical load due to two or three upper stories.

For all the tested wallets, the diagonal tension was identified by Javed et al. (2012) as the experimental failure mode. In Fig. 4, the strength criteria associated with the shear behavior (Turnsek&Cacovic or Mann&Muller) and the flexural response (fixed or cantilever conditions) are plotted; the markers (i.e., square markers for W1, circle markers for W2, and diamond markers for W3) refer to the outcome of the experimental tests. For example, in the case of W2 series, the three tested wallets in both loading directions yield six circle markers in Fig.4b,d for the pre-compression level 0.42Mpa. In particular, the Turnsek&Cacovic criterion is represented by a solid black line, whereas the Mann&Muller one by a black dash-dotted line. The flexural response considering fixed or cantilever conditions are represented in the blue dotted line, and blue dashed line, respectively. Since a double fixed set-up experiment was employed by Javed et al. (2012), the most representative flexural domain to be considered for these tests refers to the dotted dashed line. The Figs 4c, 4d are a zoom-in view of Figs 4a and 4b respectively. Views of the W1, W2b and W2c wallets at the end of testing are shown in Figs 4e, 4f and 4g respectively (Javed, 2009).

Table 2. *Geometry and pre-compression levels for a) the wallets b) selected piers*

Parameters	wallets			piers of the single room		
	W1	**W2**	**W3**	**P2**	**P3**	**P4**
length (mm)	1360	1360	1360	870	690	690
height (mm)	1660	1280	1280	2743	2743	1676
thickness(mm)	236	236	236	229	229	229
aspect ratio	1.22	0.94	0.94	3.15	3.98	2.43
Pre-compression levels (MPa)	0.71	0.42	0.64	(0.15-0.2)	(0.14-0.19)	(0.14-0.19)

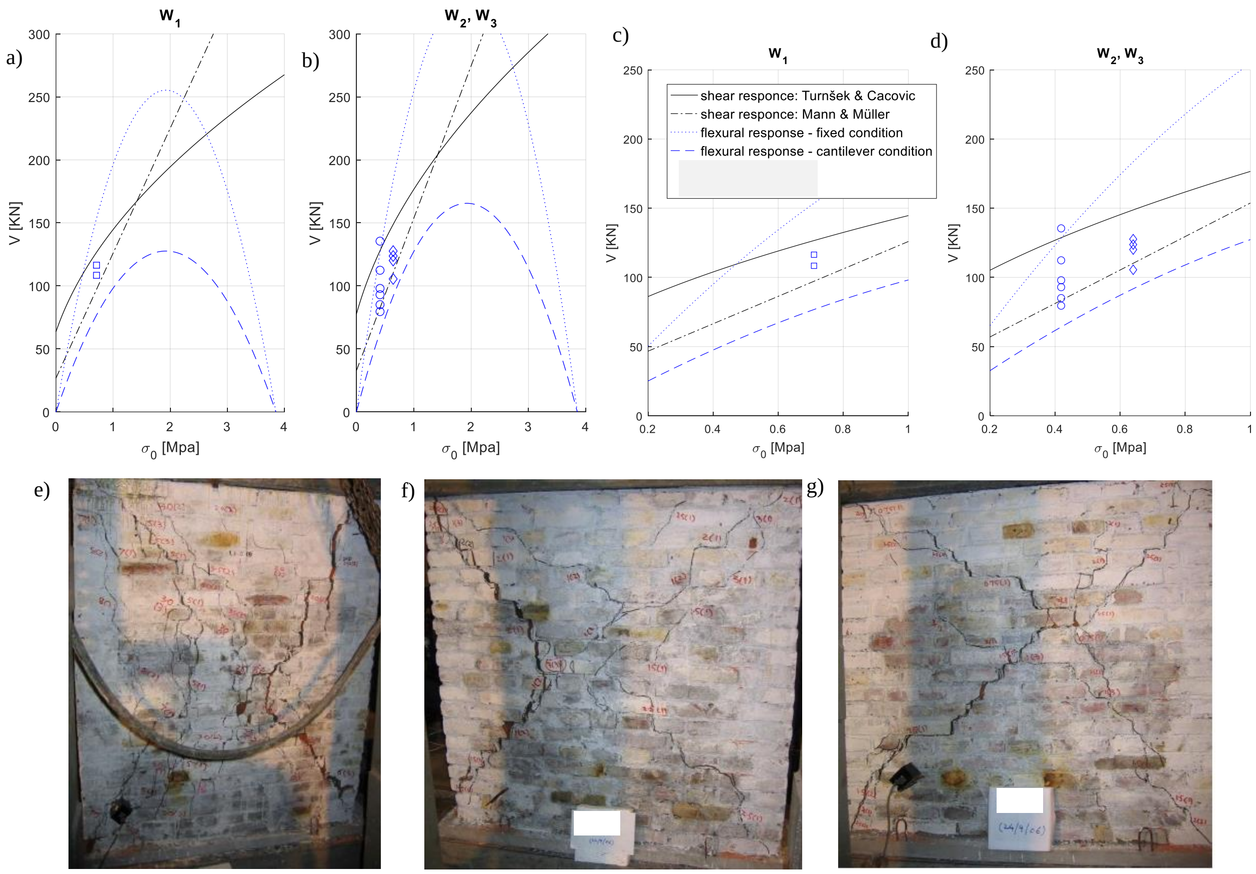

Figure 4. *Tested wallets: Analytical strength criteria (lines) and experimental data (markers)in both loading directions, for the a),c) W1, b), d) W2a, W2b, W2c, W3a and W3b, and a view of wallets e) W1, f) W2b and g) W2c at the end of testing (Javed, 2009)*

Table 3 summarizes the consistency of the activated failure mode in the experimental set with the predicted one based on the minimum from those provided by the analytical strength criteria. The analytical function consistent with the Mann & Müller criterion is the closest to the discrete experimental points (markers) for most of the cases. This appears reasonable due to the regular texture and the use of blocks more resistant than mortar joints that promote the activation of a stair-stepped diagonal crack. It is worth mentioning that the lateral strength of all the wallets was found (Javed, 2009) to be approximately the same in both loading directions except the second tested wallet of the W2 series. Those two values are the two upper circle markers at pre-compression level 0.42 (Fig. 4).

Table 3. *Failure mode summary*

Failure mode	*W1*	*W2*	*W3*	*P2*	*P3*	*P4*
experimental test	Diagonal cracking	Diagonal cracking	Diagonal cracking	Diagonal cracking	Flexural	Diagonal cracking
strength criteria	Shear (Mann & Müller, but Turnšek & Cacovic very close)	Shear (Mann & Müller)	Shear (Mann & Müller)	Hybrid mode (flexural and shear prediction very close to each other)	behavior	Shear (Mann & Müller)

For some representative piers of the single room (Shahzada et al., 2012), i.e., the P2, P3, and P4, the shear strength values V are plotted in Fig. 5a. These piers in a plan view are shown in Fig.1b and the final damage pattern of the tested structure is shown in Fig. 5b. The estimated pre-compression level is denoted with two vertical lines: one for the value on the bottom of each pier and one for the value on the top. The estimated axial loading includes the axial loading due to: a) the adjacent parts of the building; b) the dead load from the roof treatment; and c) the self-weight of the RC slab (Shahzada et al., 2012) (see Fig. 1b front view). The geometrical and the relevant estimated pre-compression levels used for the analytical equations are summarized in Table 2, and the mechanical parameters are listed in the second column of Table 1. The tensile strength of brick units is equal to 10% of the compressive strength of units. In Fig. 5a, the shear response is plotted with solid and dash-dotted black lines for both criteria proposed by Turnšek & Cacovic and Mann & Müller, respectively. The flexural response is plotted in Fig. 5a for five constraint conditions. The fixed and cantilever conditions are denoted as before with a dotted and dashed blue line, respectively. In Fig. 5a, three additional thinner lines are plotted for ξ coefficients equal to 1.2, 1.5, and 1.7 considering intermediate constraint conditions. As stated in Shahzada et al. (2012), the damage pattern of the test structure was a combination of shear and flexural cracks, with shear being the predominant failure. Since a shear response took place in many cases, the expected restraint provided by the rigid heavy RC slab is considered as an intermediate restraint between the cantilever and the fixed condition. Otherwise, since the panels are slender (Table 2), the flexural failure mode would have dominated the failure pattern. Thus, the analytical function that describes the flexural response is selected to be the one with the intermediate coefficient equal to 1.5, denoted with the orange line, i.e., the line in between the three thinner lines. Table 3 summarizes the consistency between the activated failure mode in the experimental set and the predicted one based on the analytical equations of strength criteria, as for the selected piers.

In the damage pattern of the single room, there are also many vertical cracks that mainly developed at the two ends of the in-plane walls. These cracks are due to a combined effect mainly due to poor connections among the two orthogonal walls and poor properties of used materials. Specifically, the IRA of the bricks and the compressive strength of the mortar are different from those commonly used in other parts of the developed world. The IRA is very high (i.e., 91.7 g/min/30in^2), resulting in a very weak bond between bricks and mortar and, consequently, low diagonal tensile strength and compressive strength. That is also why the cracks are developed at the interface of the mortar and bricks (Shahzada et al., 2012). Moreover, some vertical cracks were also observed at the intersection of the south wall with the east and west walls (Shahzada et al., 2012), mainly due to the poor connections between the two orthogonal walls (see Fig. 3c). This has also been verified in the experimental investigation of a confined masonry single room (Ahmed et al., 2019), with the same configuration and material properties as the considered URBM single room. In the confined masonry case these vertical cracks are eliminated.

a)

b)

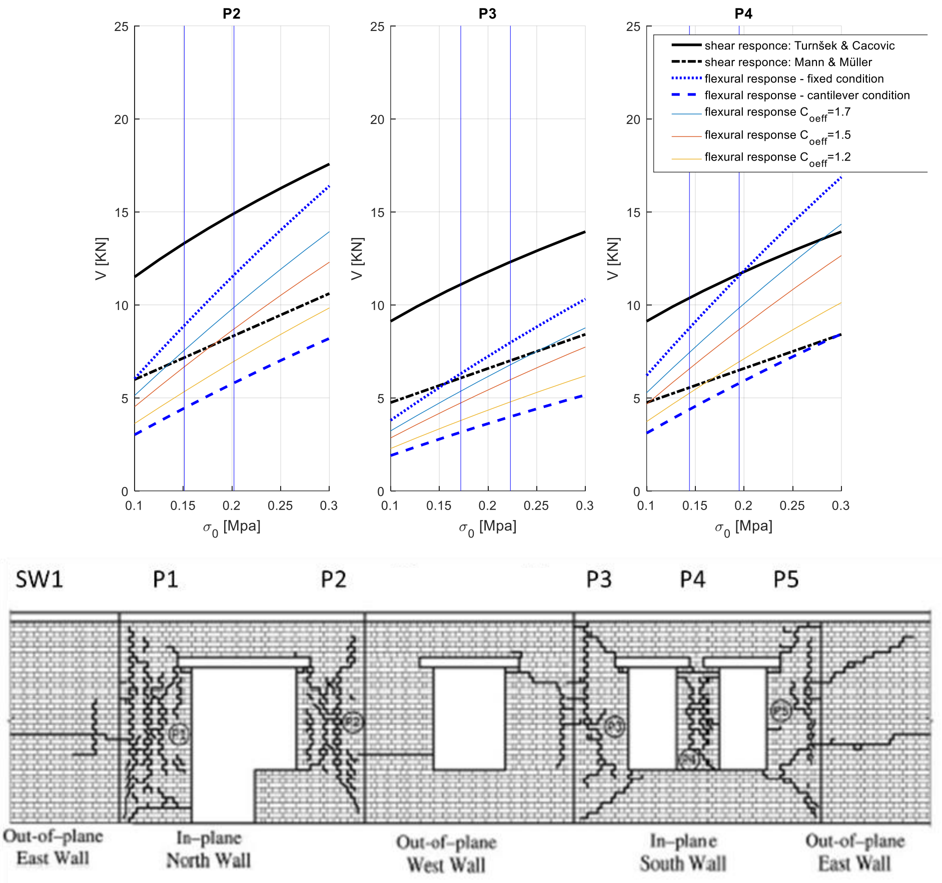

Figure 5. *Single room: a) strength criteria denoted with continuous lines for selected piers, b) the final damage pattern of the structure (Shahzada et al., 2012).*

CONCLUSIONS

A basis is provided for defining archetypes for a dominant and vulnerable building type, i.e., the non-engineered low-rise (1, 2-story) URBM residential dwellings with a heavy RC slab in Pakistan. Supported by the expert opinion about the local practices and constructions trends, this groundwork has two main pillars: the data collection and the use of analytical strength criteria combined with representative, i.e. based on the most recurring configuration and commonly used construction materials, experimental data.

This work 1) provides an overview about the salient characteristics of the considered building type, 2) defines the reference values to be adopted in the analytical equation of wall strength criteria, and 3) verifies, for the studied building type, the consistency of the failure mode predicted by the analytical strength criteria equations with the one activated in the experimental test. Among the strength criteria available in the literature to interpret the diagonal cracking, the one proposed by Mann & Müller appears the most suitable to interpret the behavior of this masonry type. Ultimately this groundwork can be employed to create structural models for building and class vulnerability assessment of the studied structures.

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