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ORIGINAL PAPER

Seismic vulnerability assessment of masonry buildings in Karachi

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Abstract
Existing masonry structures are severely affected when subjected to strong ground motions, this could be due to the lack of maintenance over the period and the presence of a significant number of seismic vulnerabilities. Restoration and retrofitting are the methods to preserve the aesthetic and overall response of these structures. Therefore, the focus of this research is to employ the non-linear static analysis approach and to investigate the seismic response of existing masonry building structures subjected to strong events. In addition, to develop the fragility functions from empirical and analytical approaches corresponding to different limit states and to assess the seismic performance of historic building stock in Karachi. For this purpose, a prototype masonry structure is modeled on a computational tool 3-Muri and processed through pushover analyses in orthogonal directions considering three different groups of masonry such as Good, Fair, and Poor to incorporate the randomness in material characteristics. Furthermore, results obtained from fragility functions were employed to generate damage matrices corresponding to different intensities of the earthquake. Moreover, for verification of results a comparison of both empirical and analytical approaches illustration is also presented.

Keywords Seismic evaluation · Masonry building · Pushover analysis · Fragility function · 3-Muri · Damage states

Introduction

Masonry constructions are generally found all over the world and are popular due to characteristics such as low cost, and the use of local materials. The main structural element of masonry structures is the wall, which is made up of a combination of masonry units and mortar and bears the majority of the structural load. Brick, block, or stone may be used to construct the units. In Pakistan, many buildings were constructed in the British regime prior to the first seismic design specifications defined in the engineering codes due to that these structures often do not satisfy the earthquake safety requirements as per recent seismic provisions.

Recent earthquakes in Pakistan have resulted in collateral damage in many parts of the country as a result of natural disasters, which has increased the need for the assessment of existing infrastructure and particularly masonry structures. In addition, Pakistan was struck by the largest earthquake in its history on October 8, 2005, with a magnitude of 7.6 and at a depth of 26 km. The epicenter of the earthquake was in Muzaffarabad's northern portion. It affected a 30,000-km^2 area, 90,000 casualties, and injured 79,000. Furthermore, more than 400,000 structures were damaged, the majority of which were made of stone masonry, concrete block masonry, and brick masonry (Maqsood & Schwarz, 2008). Based on recent reported seismic events in Pakistan different types of seismic sources are identified (Fig. 1) by Waseem et al. (2020). Accounting for these seismic sources the author performed a probabilistic seismic hazard analysis (PSHA) and proposed a seismic hazard map for Pakistan corresponding to different return periods (Fig. 2).

This study revealed that Pakistan's center and northern regions built environment is more vulnerable and susceptible

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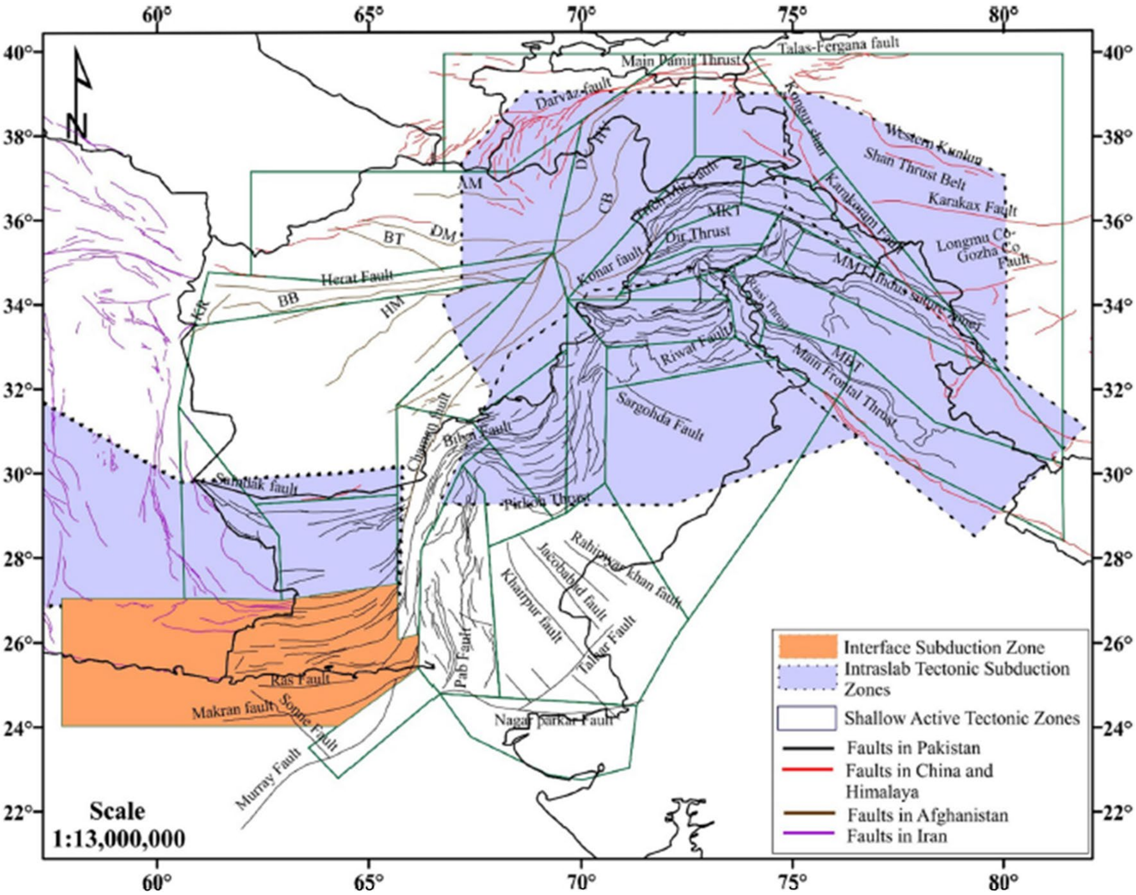


Fig. 1 The seism tectonic model (source model) of Pakistan considered in the Pakistan Seismic Hazard Assessment (PSHA) (Waseem et al., 2020)

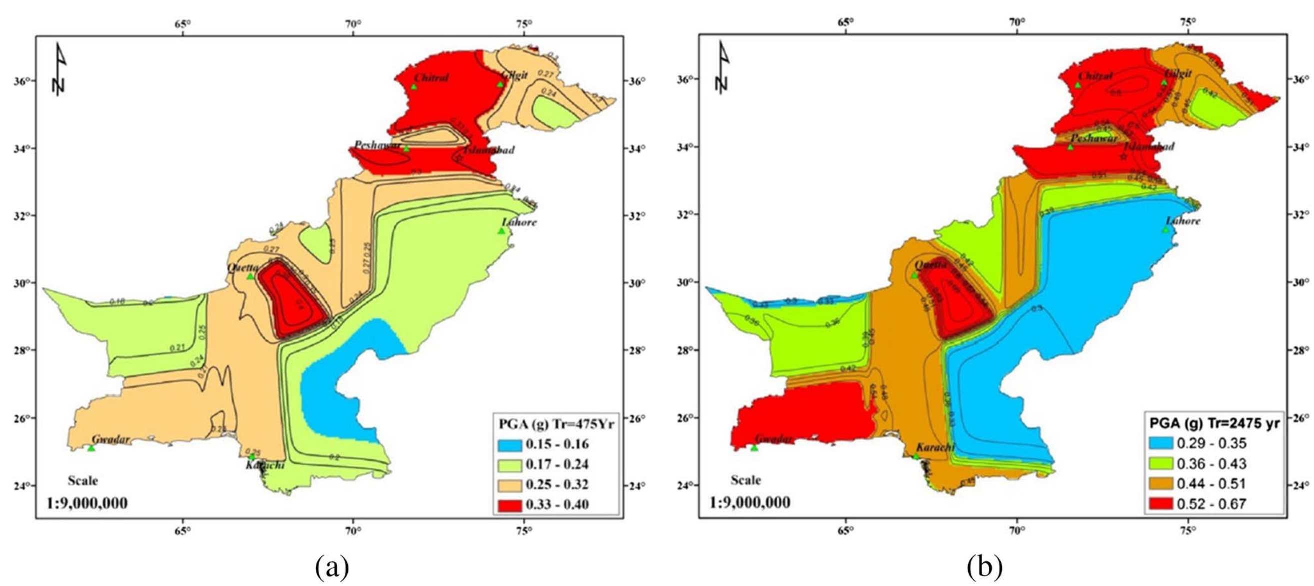


Fig. 2 Seismic zonation maps for different return periods, **a** PGA (g) for 475 years and **b** PGA (g) for 2475 years (Waseem et al., 2020)

to damage with a peak ground acceleration (PGA) of 0.4 g. Furthermore, as shown in the figures the PGA values for Balakot (Islamabad, Peshawar, Chitral), Gilgit, Karachi, and Gwadar are 0.36 g, 0.34 g, 0.26 g, and 0.29 g for 474 years return period (RP), respectively.

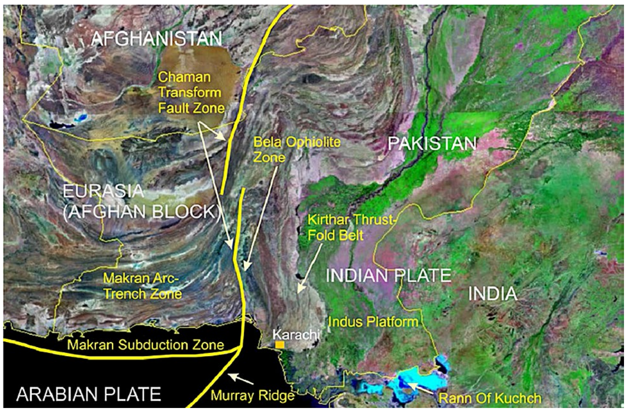


Fig. 3 Tectonic setting of Karachi with respect to plate boundaries and principal tectonic zones in southern Pakistan and offshore (Waseem et al., 2019)

Karachi, Pakistan's largest and most populous metropolis with a population of over 18 million people living in an active seismic and tectonic setting. Karachi had a relatively safe seismic history over the previous 175 years, however, it is located in plate boundary faults, triple junctions, and tectonic conditions similar to those found in the southern United States as shown in Fig. 3, considered as the most active tectonic setting in the world (Waseem et al., 2019).

Karachi is a global metropolis with many historic buildings (Fig. 4) dating back to the nineteenth century that was established and rebuilt during the British regime. Due to decay over time, a lack of maintenance, and the apathy of building owners, the current state of such structures has deteriorated. Past records showed that in Karachi around 85% of the structures were damaged and 15% were demolished, these heritage structures could be strengthened, repaired, or restored in their original state by employing the state-of-the-art retrofitting schemes reported in the literature.

Therefore, efforts should be made to preserve the existing building stock wherever possible. Following this, significant research studies were conducted to assess and analyze the seismic susceptibility of existing structures to retrofit them. Jasieńko et al. (2016) investigated a variety

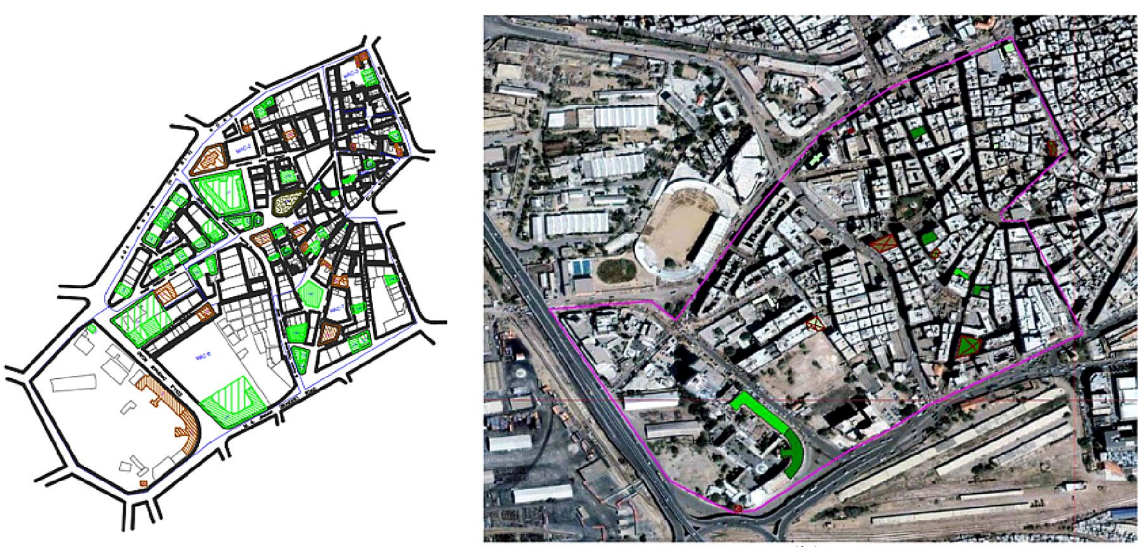
Fig. 4 Heritage building stock in Karachi

of structural materials for strengthening historic masonry structures. Redweik et al. (2017) performed 3D modeling of Lisbon city to check the seismic vulnerability of the whole city on City Engine 2015 (Mueller et al., 2008). The results can be seen through building geometry and colors as defined in EMS98 (European, 1998). McBean (McBean, 2015) tracks the dynamic response of the Royal Adelaide Hospital using the Finite Element Method (FEM) (Pradhan & Chakraverty, 2019) in ETABS. Sözen and Çavuş (2019) modeled a historical structure on SAP2000 and access the structural integrity and increment in seismic performance against the lateral loads. Cardoso et al. (2005) developed a numerical model of an old building on SAP2000, and track the nonlinear response against earthquake loading. Ajmal (2012) investigated the response of a historic building not designed for seismic loading and damaged by several strong earthquakes in past. Sattar (2013) developed a multi-scale modeling approach and simulate the collapse behavior of the masonry infill interacted with the reinforced concrete (RC) frames. Ademovic et al. (2013) analyzed the behavior of typical masonry buildings by the FEM method on 3-Muri and DIANA (V., 2020). This study concluded that the accuracy of results is directly dependent on the level of uncertainties used in the analysis. Another study by Pegon et al. (2001) includes the 2D and 3D modeling of built heritage structures to get the results from the nonlinear analysis. The model was developed for observing the global response of as-built structure characteristics. Several other studies (Chieffo et al., 2020; Mosoarca et al., 2020; Onescu et al., 2021) were being carried out all over the world which show the significance of seismic assessment and retrofitting of heritage structures.

In the current study, a G + 4-story masonry building part of the heritage stock in Karachi had been selected. The building was modeled on 3-Muri (S.T.A.-Data), and analyzed through the state-of-the-art pushover analysis approach to depict the non-linear behavior and clear damage pattern at component and global levels. Unreinforced masonry walls are generally prone to failure in the Out-of-Plane direction. However, in the current study, only the global response of the prototype building has been investigated. After the development of the capacity curves, fragility functions have been developed considering PGA as an intensity measure (IM) corresponding to different limit states (LS) such as slight (S), moderate (M), extensive (E), and complete (C). For verification and comparative analysis, empirical fragility functions are also derived with the help of HAZUS®MH MR4 technical manual (FEMA 2003a, b). Furthermore, damage matrices developed for PGAs correspond to different return periods such as 50, 100, 475, 1000, and 2500 years.

Case study structure

In Karachi, heritage building stocks are constructed in a cluster format, and to understand the behavior of these structures in the current study, a cluster of Macchi Miani Quarter has been identified with a layout plan shown in Fig. 5. The boundaries of Macchi Miani Quarter lie between M. A.



(a) (b)

Fig. 5 Layout plan of Macchi Miani Quarter **a** sketch, and **b** Google image (Cell, 2019)

Table 1 General information of building (Cell, 2019)

Typ. story height	3.50 m
Size	24.5 m × 18.77 m
Soil type	D
Building usage	Commercial plus residential building

Jinnah (Bunder) Road on the south-east, Adamjee Dawood Pota (Rampart) Road on the north-east, and Aga Khan (Harris) Road on the north-west and Edulji Dinshaw Road on the south-west.

The building heights variation within Macchi Miani Quarter indicates that the heights of all historic buildings range from ground to ground plus four stories at the most. Therefore, a G + 4 story residential building named Gulshakar Manzil, located at Haji Abdul Shakoor Road Kharadar Saddar Town, at latitude and longitude of 24° 50′ 59.43″ N and 60° 59′ 30.97″ E, respectively, has been selected as a prototype masonry building. Table 1 shows the significant characteristics of the prototype building with the plan and front elevation details shown in Fig. 6.

The identified case-study building (Fig. 6a) is a preeminent reprehensive of the building stock in Karachi and made up of stone (Calcium Silicate Block) masonry with lime mortar. The structure is partially maintained and used for commercial at the ground (Fig. 6b) and residential purposes in upper stories (Fig. 6c). The front façade (Fig. 6d) with

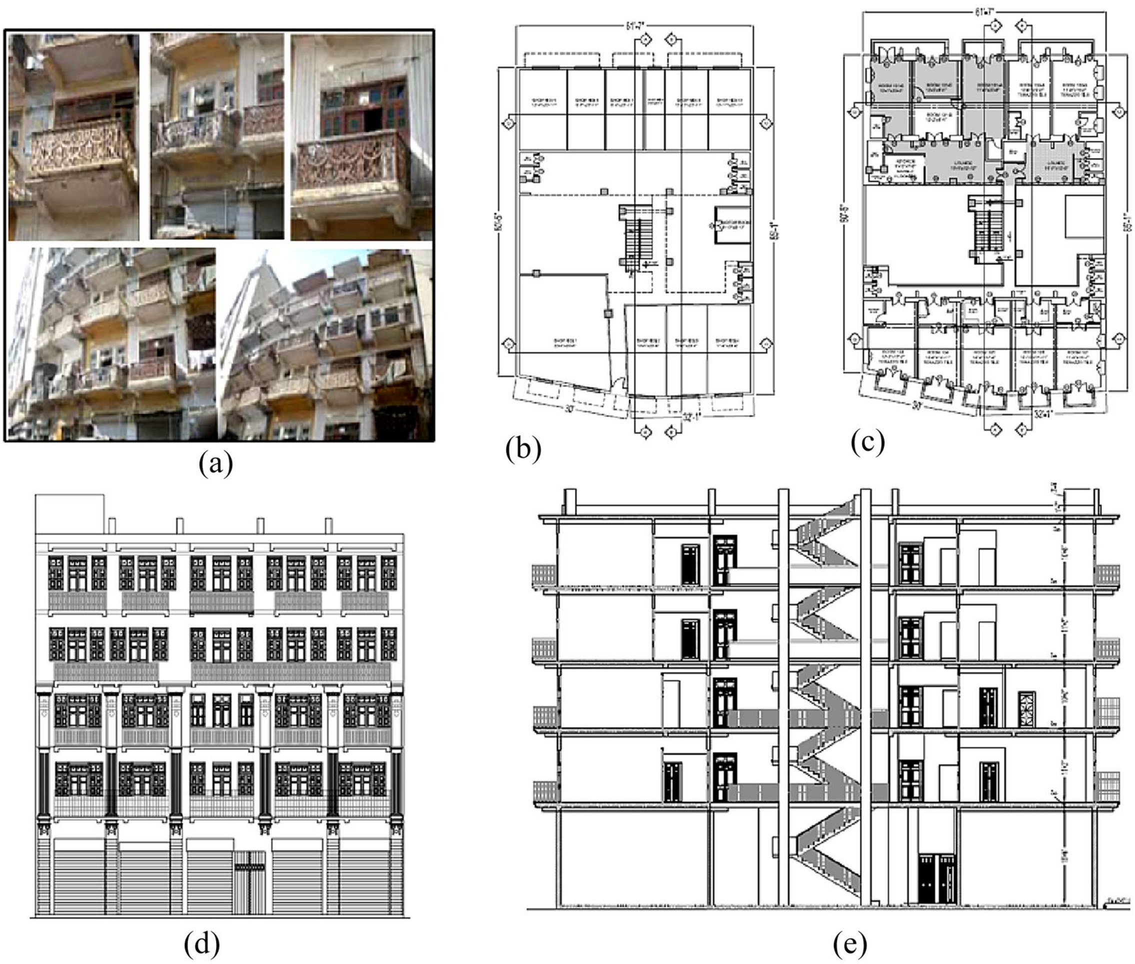


Fig. 6 Gulshakar Manzil **a** 3D view, **b** floor plan at the ground, **c** typical floor plan in upper stories, **d** front elevation and **e** section elevetion in longer direction (Cell, 2019)

balconies is parallel to the roadside and in the shorter direction (*X*-direction) of the building. Furthermore, the building has substantial structural walls with the typical thickness of 30 cm in the longer direction (*Y*-direction) shown in Fig. 6e and few walls are located in the transverse or shorter direction.

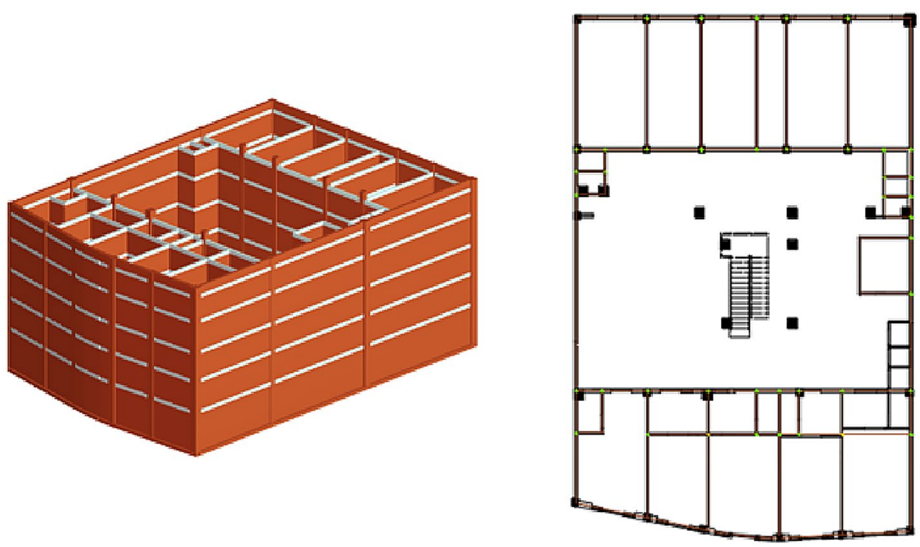
Computational modelling

In the current study, to evaluate the seismic behavior of existing masonry structures a state-of-the-art computational tool 3-Muri is employed. 3-Muri is a computer program specifically designed for seismic evaluation of masonry structures developed by Lagomarsino et al. (2013). 3-Muri works on an Equivalent frame modeling approach that includes several macro models and evaluates the structure to an extent of refined results through analysis. The equivalent frame modeling approach is the most widely used approach for the evaluation of masonry structures as it provides a realistic response for the 3D structure, particularly at the global level (Federal Emergence Management Agency 2000).

3-Muri allows modeling of any regular or irregular-shaped building structure in 2D and simultaneously converting it into 3D for clear and error-free modeling shown in Fig. 7. Before starting modeling, units were set considering the US customary unit system. For modeling of structure, a DXF format file is required to be exported to 3-Muri. So, a blend of all architecture plans was created on AutoCAD and a single file was exported to 3-Muri for modeling the entire structure. After exporting the DXF file walls were assigned (traced) along the walls as indicated in the architectural plan by using the insert wall tab. Several options were used during the modeling of the structure including trim, extend, fillet, stretch and delete walls. After tracing the walls structural material and elements were assigned to the walls by using the structure tab. From the list of different materials provided in the 3-Muri material library, Muratura (masonry) was selected as a modeling material for the structure. In Karachi heritage building stock, the base material consists of calcium silicate blocks with lime mortar.

The value for each parameter was set to default and Turnšek Cačovic was selected as the constitutive model because this structure is an existing building and Turnšek Cačovic is typically employed constitutive model for the analysis of existing structures.

After defining material property walls were assigned considering the attributes of the as-built structure. Characteristics of each wall were defined by selecting each of them and the thickness of the masonry panel and width of the beam (pad beam) were also considered in modeling with variant characteristics. The concrete cover was assigned as 40 mm (1.58 in) and the bar used for the stirrup was a 10 mm



(a) (b)

Fig. 7 Gulshakar Manzil **a** 3D view and **b** plan of the FEM model generated in 3-Muri

Table 2 Modal period and mass participation

Mode	*T* (s)	Mx (%)	My (%)	Mz (%)
1	0.24390	84.15	0.00	0.00
2	0.21443	0.01	85.27	0.01
3	0.18308	1.18	0.10	0.00
4	0.09210	11.29	0.01	0.18
5	0.08117	0.03	5.02	43.25
6	0.07851	0.00	4.62	46.42
7	0.06399	0.48	0.00	0.00
8	0.06204	1.07	0.26	0.04
9	0.05881	0.09	1.92	0.10
10	0.05245	0.16	0.03	0.00

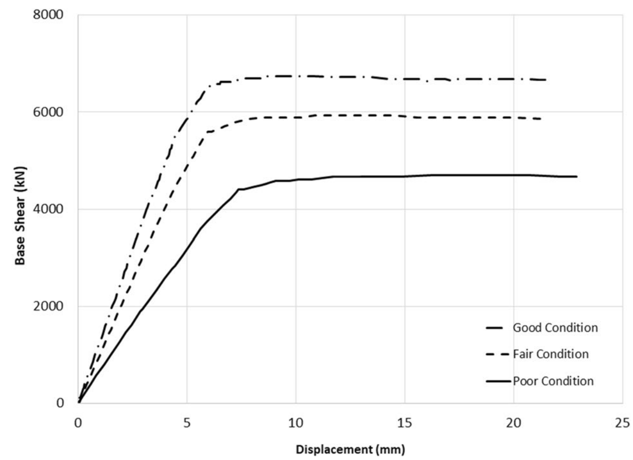


Fig. 8 Capacity curves with different masonry characteristics

diameter bar. Masonry foundation was assigned to the structure with the modulus of subgrade reaction considered as 588 N/cm^3 (3743.13 kips/ft^3).

Columns were modeled from the base to the last level of the building, typical sizes of columns were considered i.e., 350 mm × 350 mm (14 in × 14 in) and 500 mm × 500 mm (20 in × 20 in), respectively. In Karachi, most of the masonry buildings have wooden floor systems resting on masonry walls. These floor systems were severely damaged due to the lack of maintenance and later on replaced by concrete floor slabs with RC pad beams. Therefore, rigid slabs were assigned to the entire structure with mass source including the self-weight, 1.91 KN/m^2 (40 Psf) as additional dead load and 0.957 KN/m^2 (20 Psf) as finishes load.

A 3D structure can be processed through nonlinear static and dynamic analyses on 3-Muri with an optimal

Table 3 Default lower-bound masonry properties (Federal Emergence Management Agency, 2000)

Property	Masonry condition		
	Good (MPa)	Fair (MPa)	Poor (MPa)
Compressive strength (f'_m)	6.20	4.14	2.07
Elastic modulus in compression	$550f'_m$	$550f'_m$	$550f'_m$
Flexural tensile strength	0.14	0.07	0
Shear strength			
Masonry with a running bond lay-up	0.19	0.14	0.09
Fully grouted masonry with a lay-up other than running bond	0.19	0.14	0.09
Partially grouted or un-grouted masonry with a lay-up other than running bond	0.075	0.055	0.034

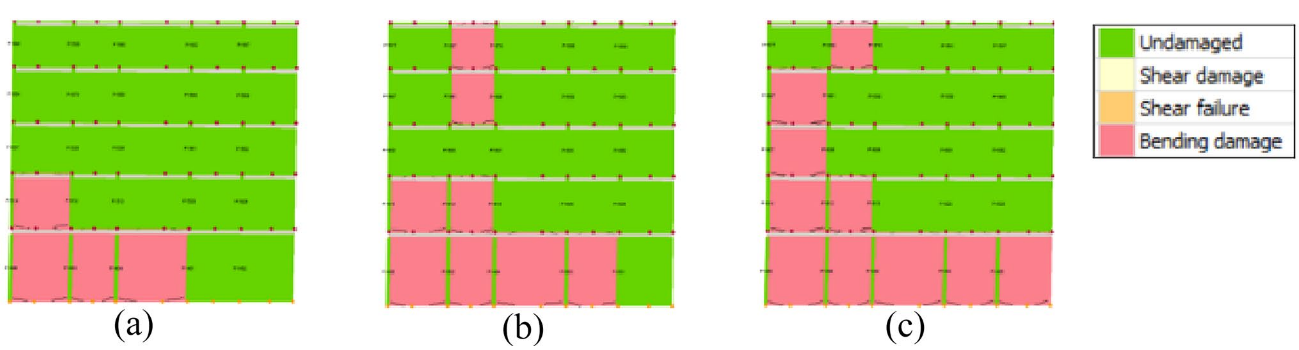


Fig. 9 Damage patterns at yield displacement with **a** good, **b** fair and **c** poor masonry in a shorter direction wall pier

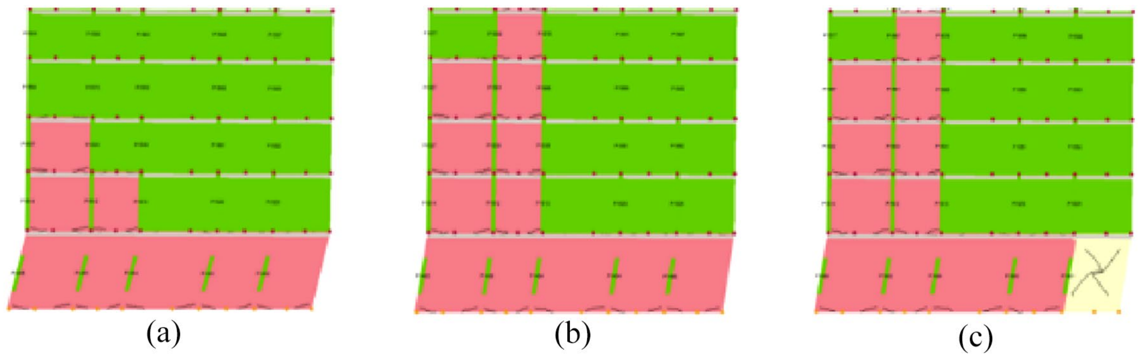


Fig. 10 Damage patterns at ultimate displacement with **a** good, **b** fair, and **c** poor masonry in a shorter direction wall pier

compromise between accuracy and computational burden. 3-Muri generally performs 24 pushover analyses in four directions; the positive and negative *x*- and *y*-directions with two different load patterns over the height of the structure. The load will be either distributed proportional to the floor masses or proportional to the linearized first Eigenmode of the structure. In the current study, only capacity curves were generated in a weaker direction and later on used to develop fragility functions.

Global static verification analysis

3-Muri performs global static verification through vertical load check and the check of slenderness ratio. The permissible limit for slenderness ratio is either less or equal to 20. Several walls (piers) failed in this check for the case-study building. The load factor (Nd/Nr) for most of the piers was undefined because of slenderness ratio was not verified for such walls.

Modal analysis

The identified building structure is analyzed through modal analysis for 10 different modes. 3-Muri shows the modal period and the corresponding mass participation in the *x*, *y*, and *z* directions shown in Table 2.

Capacity curve with material uncertainties

To incorporate randomness in material characteristics and due to the unavailability of test results, the analysis performed herein for three different shear strengths considering uncertainties in material properties. The lower, upper and mean values are obtained from lower-bound masonry properties defined by FEMA-356 (Federal Emergence Management Agency, 2000) shown in Table 3 termed as good, fair, and poor quality masonry. Based on these material characteristics capacity curves are developed and shown in Fig. 8. Furthermore, the building configuration shows that the case-study structure is weaker in the shorter direction (*x*-direction) due to the lack of an adequate number of the load path. Therefore, the damage patterns observed for a specific wall pier only in the *x*-direction and particularly reported at yield (d*y*) and ultimate (d*u*) displacements limit states with different types of masonry (Fig. 9). Results show that at ultimate displacement limit state soft-story collapse mechanism observed in all three types of masonry and shear failure found particularly with poor masonry wall pier at the base (Fig. 10).

Table 4 Structural fragility curve parameters: pre-code seismic design level (FEMA, 2003a, b)

Building properties	Spectral displacement (mm)							
Type	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
URML	8.12	29.20	16.5	30.2	41.48	30.5	96.0	30.0
URMM	**12.7**	**25.14**	**25.65**	**24.63**	**64**	**22.86**	**149.352**	**22.35**
MH	9.65	28.19	19.56	27.94	58.42	24.13	170.7	24.63

The bold values herein are spectral displacements used for the identified case-study building

Table 5 PGA (g) values for good, fair, and poor masonry characteristics

Damage states/masonry conditions	Good	Fair	Poor
Slights	0.187	0.166	0.136
Moderate	0.220	0.199	0.158
Extensive	0.533	0.518	0.361
Complete	1.213	1.029	0.625

Results and discussions

Seismic fragility functions

Fragility curves are one of the widely accepted approaches to represent the vulnerability assessment information, specifically when multiple sources of uncertainties exist, for example, seismic demands, soil-structure interaction, structural characteristics, etc. Fragility curves (FEMA, 2003a, b) are lognormal distributions that show the probability of exceedance of damages corresponding to different limit states such as slight, moderate, extensive, and collapse or complete in a structure in response to ground shaking. Fragility curves portray the probabilistic information for the correlation between limit states and the seismic demands. Fragility functions can be developed through different approaches such as empirical, judgmental, analytical, and hybrid approaches. In the current study, analytical and empirical approaches are employed to develop these functions considering PGA as an IM and spectral displacement (Sd) as an engineering demand parameter (EDP) corresponding to different limit states. PGA is considered an efficient and sufficient IM particularly used in the vulnerability assessment of low-rise buildings. Furthermore, in the current study, fragility functions developed corresponding to different types of limit states such as *slight* representing diagonal or stair-step hairline cracks in masonry, *moderate* represent larger diagonal cracks in masonry, *extensive* represent severe diagonal cracking and some wall may have fallen and *complete* represent 15% of the total masonry area completely damage or collapse.

Table 6 PGA (g) for different return period (Ahmed et al., 2019)

Return period (years)	PGA (g)
50	0.08
100	0.13
475	0.23
1000	0.26
2500	0.41

Fragility functions by analytical approach

To quantify the building response, damage states employed herein adopted from the HAZUS methodology (FEMA, 2003a, b). To this end, demand spectra have been used obtained from ASCE 7–16 and overlapped on the capacity curve by scaling up or scaling down corresponding to different damage states in terms of engineering demand parameters (EDPs) i.e., spectral displacements. These spectral displacements were reported in the HAZUS manual for different typologies, amongst these EDPs spectral displacements extracted for unreinforced midrise masonry structure (URMM) corresponding to pre-code seismic design level reported in Table 4. After overlapping the demand spectra for a particular damage state, the corresponding PGA of the scaled spectrum is considered the best estimate of PGA (μ), these PGAs are listed in Table 5 for different types of masonry i.e., good, fair, and poor.

Finally, fragility curves have been developed shown in Fig. 11 using the lognormal distribution function taking spectral acceleration within the range of 0.01–0.3. The standard deviation of the natural logarithm of spectral acceleration of a particular damage state referred to as β_{ds}, was taken as 0.64, as mentioned in HAZUS®MH MR4 (FEMA, 2003a, b).

Table 7 Equivalent-PGA structural fragility: pre-code seismic design level (FEMA, 2003a, b)

Building type	Median equivalent PGA (g) and Log standard deviation (Beta)							
	Slight		Moderate		Extensive		Complete	
	Median	Beta	Median	Beta	Median	Beta	Median	Beta
URML	0.13	0.64	0.17	0.64	0.26	0.64	0.37	0.64
URMM	**0.09**	**0.64**	**0.13**	**0.64**	**0.21**	**0.64**	**0.38**	**0.64**
URMH	0.08	0.64	0.11	0.64	0.18	0.64	0.34	0.64

The bold values herein are spectral displacements used for the identified case-study building

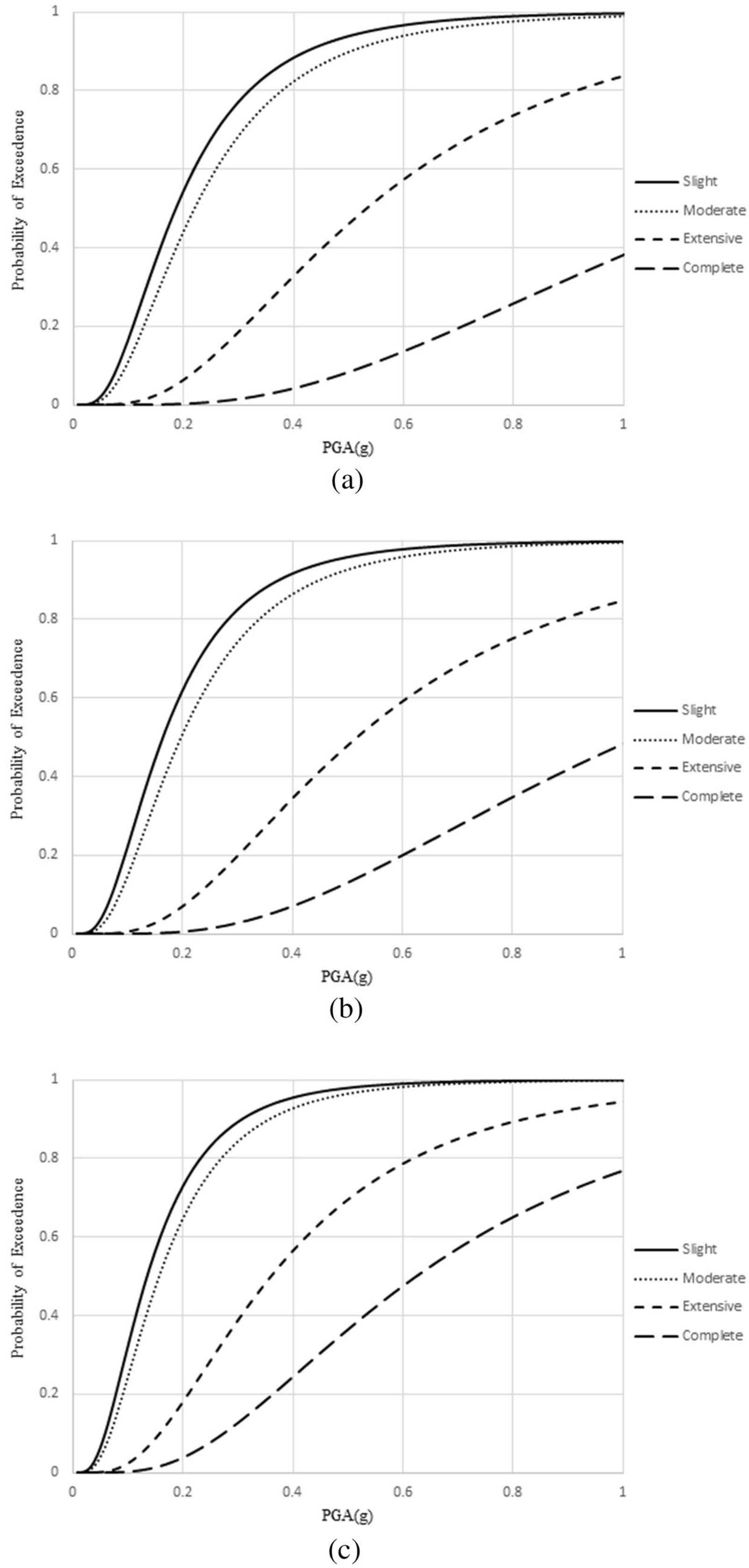


Fig. 11 Fragility curves for **a** good condition, **b** fair condition and **c** poor condition masonry

In vulnerability assessment, the next step is to develop the damage matrix for a particular typology of structure based on the fragility function corresponding to different intensities of the earthquake. Therefore, herein Table 6 shows the values extracted from the seismic hazard map of Pakistan corresponding to different return periods (Ahmed et al., 2019).

For the aforementioned intensity measures, damaged matrices have been developed corresponding to different limit states, shown in Fig. 12. These matrices show the percentage of buildings affected by a specific intensity of an earthquake. These damage matrices are useful for disaster management authorities to plan disaster mitigation strategies according to the intensity of the hazard and the vulnerability associated with building typologies.

Fragility functions by empirical approach

In empirical fragility functions, the relationship between seismic excitation and limit states is usually established by employing post-earthquake loss statistics. In the current study, the empirical fragility function parameters have been obtained from HAZUS-MR4 (FEMA, 2003a, b) technical manual. HAZUS-MR4 is a vulnerability assessment tool to help in estimating the risks of natural disasters like tsunamis, earthquakes, floods, fire, etc. In this manual, there are 36 building typologies along with different heights such as low-rise, mid-rise, and high-rise fragility functions parameters are reported. In the current study, the prototype building belongs to the category of unreinforced mid-rise masonry structure (URMM), hence URMM fragility functions parameters have been selected as shown in Table 7.

The aforementioned fragility function parameters employed herein and fragility curves have been developed (Fig. 13) for unreinforced mid-rise masonry structures.

Based on fragility functions, a damaged matrix has been developed corresponding to different intensities of earthquakes shown in Fig. 14.

Comparative analysis of analytical and empirical approaches

For verification of results obtained from the analytical approach, a comparative analysis has been conducted between two different approaches. Before comparison, an average matrix (Fig. 15) has been developed from matrices obtained corresponding to good, fair, and poor masonry in the analytical approach. Finally, results obtained corresponding to the return period of 475 years have been selected from both the approaches and compared.

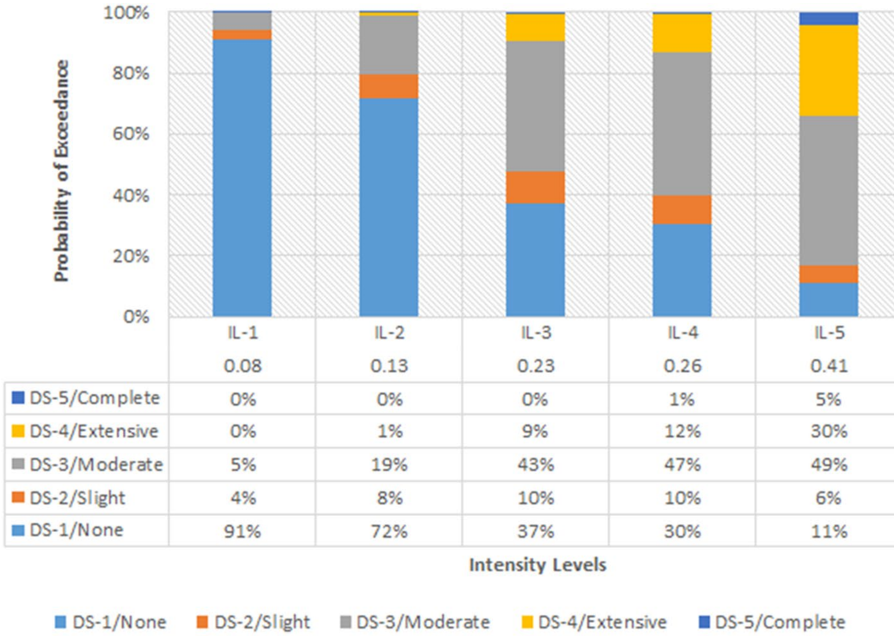
Results show that at an intensity level of 0.23 g, there is no damage observed in 30% of structures from the analytical, and 7% from the empirical approach. Furthermore, 9% of structures from the analytical, and 12% from the empirical approach are slightly damaged and belong to DS-2. Moreover, 47% of structures from the analytical, and 26% from the empirical approach are moderately affected. In addition, 12% of structures from the analytical, and 34% from the empirical approach are in DS-4 or extensively affected. Whereas, 2% from the analytical, and 22% of structures from the empirical approach are completely collapsed.

There is a significant difference in results shown in Table 8 obtained from analytical and empirical approaches, this randomness in results is due to the variation in building geometrical and material characteristics and also the variation in site-specific seismic hazard characteristics accounted in HAZUS technical manual as compared to the seismology of Karachi.

Conclusions and recommendations

In recent years, more and more emphasis has been placed on sustainable and resource-conserving construction. For this reason, efforts should be made to preserve the existing building stock wherever possible. Since many buildings in Pakistan and particularly in Karachi were constructed in the British era, these structures often do not fulfill the seismic safety requirements according to the recent modifications reported in building codes. Therefore, in the current study, the seismic performance of a G + 4 story unreinforced mid-rise masonry (URMM) building is investigated. The global response of the 3D structure is investigated employing 3-Murie with state-of-the-art nonlinear static seismic assessment methodology. Furthermore, to incorporate the epistemic uncertainties three different groups of masonry based on their strengths such as good, fair, and poor quality masonry considered as base case material in the prototype structure. After the development of the capacity curves, fragility functions have been developed considering PGA (g) as an intensity measure (IM) corresponding to different limit states (LSs) such as slight (S), moderate (M), extensive (E) and complete (C). For verification and comparative analysis, empirical fragility functions are also derived from the

Fig. 12 Damage matrices **a** for good condition, **b** fair condition, and **c** poor condition masonry structures



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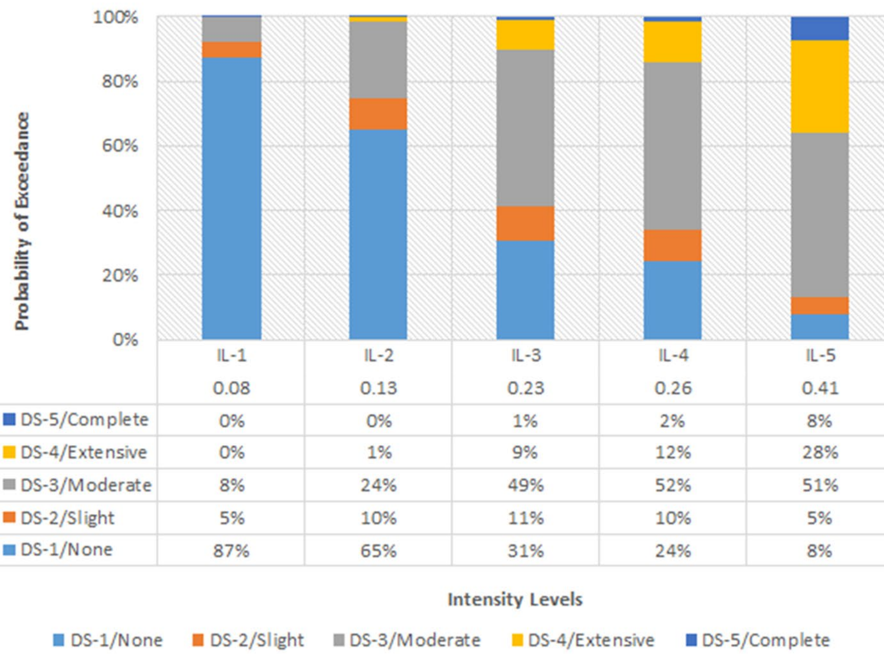
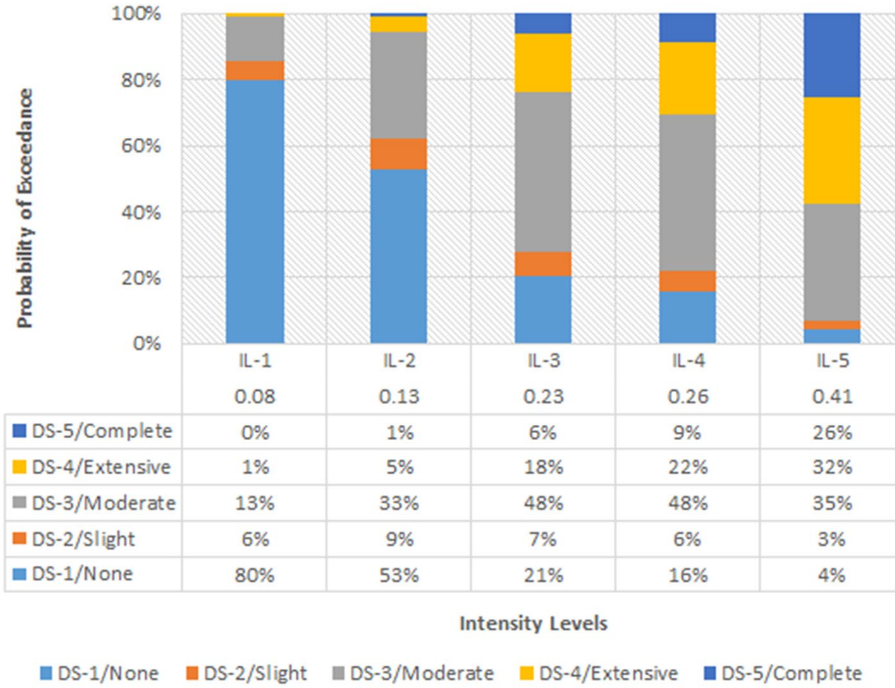


(b)

HAZUS methodology. Moreover, damage matrices developed for PGAs correspond to different return periods such as 50, 100, 475, 1000, and 2500 years. Comparative analysis shows that there is a significant difference in results obtained from analytical and empirical approaches, this randomness in results is due to the variation in building geometric and material characteristics and also the variation in site-specific seismic hazard characteristics

Fig. 12 (continued)



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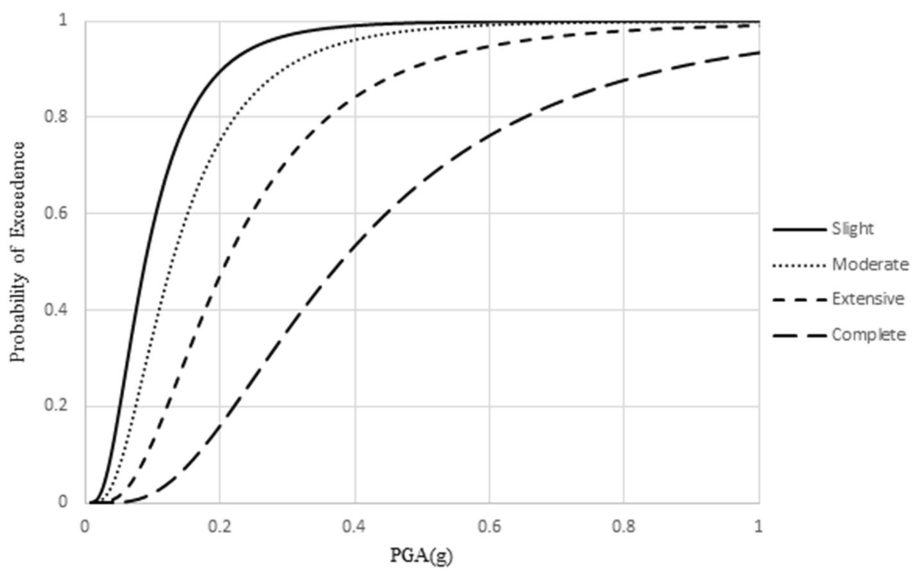

Fig. 13 Fragility functions from empirical approach (FEMA, 2003a, b)

Table 8 Comparison of analytical and empirical approaches for 475-year-return period

Damage state	Analytical approach %	Empirical approach %
None	30	7
Slight	9	12
Moderate	47	26
Extensive	12	34
Complete	2	22

accounted in the HAZUS technical manual as compared to the seismology of Karachi. For future research and a further improvement in results, it is recommended to incorporate in detail the epistemic and aleatory uncertainties in the analysis. Furthermore, incremental dynamic analysis needs to be employed as it is more rational for both irregular and high-rise structures.

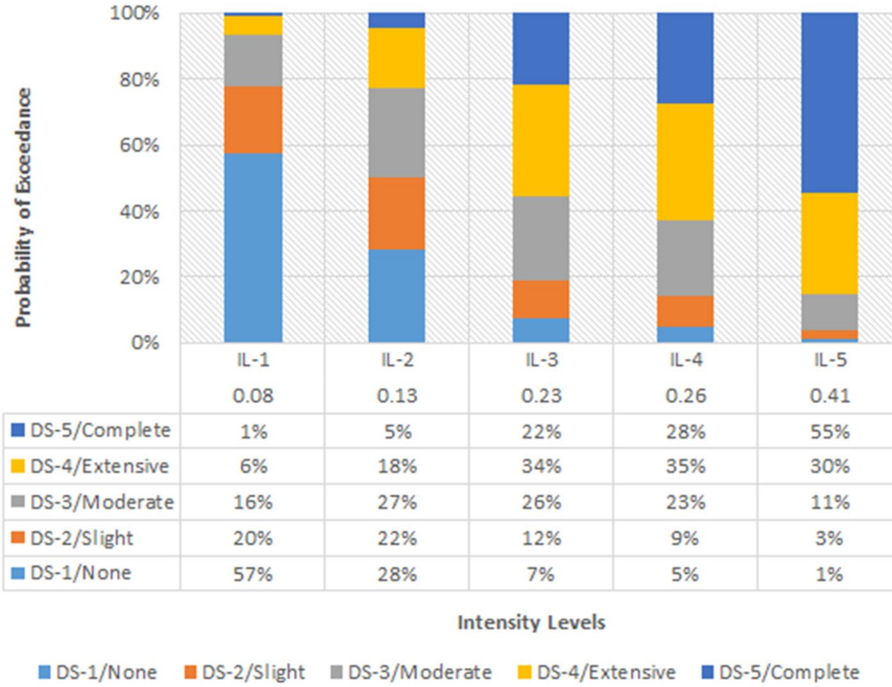

Fig. 14 Damage matrix for different return-period of earthquakes based on an empirical approach

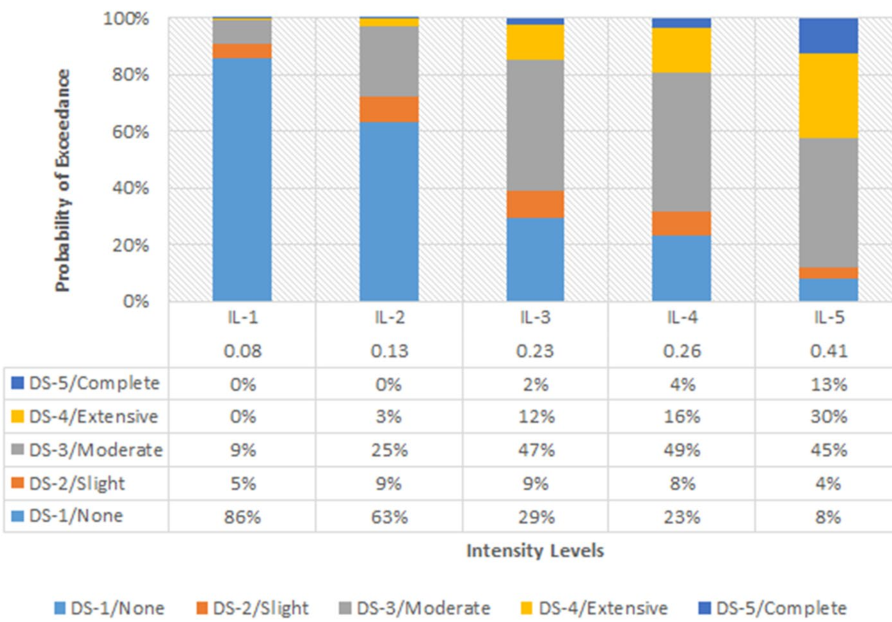

Fig. 15 Average damage matrix from the analytical approach

Author contributions AFM: float the concept, structure the paper and supervised the analytical modeling and paper writeup. RAK: review the whole work and share the guidelines about the paper writeup. EBF: develop the detailed computational model of the existing structure. EAS: develop the results with reasoning under the supervision of AFM. EMM: develop the results with reasoning under the supervision of AFM.

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Declarations

Conflict of interest The authors declare no competing interests.

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