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Seismic safety assessment of unreinforced masonry low-rise buildings in Pakistan and its neighbourhood

K. A. Korkmaz

Suleyman Demirel University, Civil Engineering Department, Isparta, Turkey

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Abstract. Pakistan and neighbourhood experience numerous earthquakes, most of which result in damaged or collapsed buildings and loss of life that also affect the economy adversely. On 29 October, 2008, an earthquake of magnitude 6.5 occurred in Ziarat, Quetta Region, Pakistan which was followed by more than 400 aftershocks. Many villages were completely destroyed and more than 200 people died. The previous major earthquake was in 2005, known as the South Asian earthquake (M_w=7.6) occurred in Kashmir, where 80 000 people died. Inadequate building stock is to be blamed for the degree of disaster, as the majority of the buildings in the region are unreinforced masonry low-rise buildings. In this study, seismic vulnerability of regionally common unreinforced masonry low-rise buildings was investigated using probabilistic based seismic safety assessment. The results of the study showed that unreinforced masonry low-rise buildings display higher displacements and shear force. Probability of damage due to higher displacements and shear forces can be directly related to damage or collapse.

1 Introduction

Earthquakes frequently hit different regions in Pakistan and its neighbourhood (see Fig. 1). On 29 October, 2008, a magnitude of 6.5 earthquake and more than 400 aftershocks hit Ziarat, Quetta Region, the provincial capital of Baluchistan, Pakistan. The Quetta region is one of the popular resort regions in Pakistan (see Fig. 2). This earthquake was responsible for more than 200 deaths, most of the existing buildings were collapsed, many villages were completely destroyed and left more than 40 000 people homeless. The previous major earthquake, also known as the South Asian earthquake, occurred in Kashmir in 2005 and 80 000 people died. Quetta was flattened in 1935 by an earthquake that killed 30 000 people.

The vulnerability of urban areas to natural disasters in places like Pakistan and its neighbourhood has attracted significant attention among practical engineers in industry and researchers in academia (Mitomi et al., 2000). Although much research has been dedicated to prevention of at least reduction of disaster damage and loss, a satisfactory level has not been achieved. The utmost importance to achieve an effective solution is the understanding of the real structural behaviour. Time history analysis is one of the accurate methods for reliable definition of the structural behaviour of building stock in the region. Various researchers have studied damage assessment by using different applications for this region (Khan and Khan, 2008; Lisa et al., 2005; Naseer et al., 2007; Zare et al., 2008).

This study focuses on seismic safety assessment of unreinforced masonry low-rise buildings in Pakistan and the neighbourhood using multiple methodologies. The seismic safety of unreinforced masonry buildings that dominate the building inventory in the region was investigated by multiple approaches. Four different representative buildings were modelled to demonstrate the building stock in the region. Nonlinear time history analysis and probabilistic based seismic assessment analysis were performed on the representative buildings. The analysis results showed that unreinforced masonry low-rise buildings present higher displacements, shear forces and probability of damage that can be directly related to damage or collapse.

Correspondence to: K. A. Korkmaz
(armagan@mmf.sdu.edu.tr)

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Fig. 1. Map of Pakistan and its neighbourhood.

2 Seismicity of Pakistan and its neighbourhood

Pakistan and its neighbourhood are prone to earthquakes. A high frequency of earthquakes has been experienced, resulting in loss of life and property destruction. According to records, this region sits in a moderate to a high seismic risk level. Parts of the North West Frontier Province, in the vicinity of Quetta and along the border with Iran, are in the high risk areas. Historically, earthquakes in the M_w 7.0 range have been experienced in Baluchistan and along the border with Afghanistan and India (ASC, 2009). The seismic map of Pakistan and its neighbourhood is given in Fig. 3 and the major earthquakes are given in Table 1 (GSP, 2009; Zaré et al., 2008; Khan and Khan, 2008). Earthquakes with a magnitude of $M>5$ in between 1973 and 2009 are depicted in Fig. 4. The Seismic hazard map is shown in Fig. 5 (USGS, 2009).

Table 1. Major earthquakes in Pakistan.

Date	Died People	Affected People	Damage ($)
31/5/1935	35 000	–	N/A
27/10/1945	4000	–	N/A
28/12/1974	4700	–	3 million
31/01/1991	5000	–	N/A
8/11/2005	73 338	2 869 142	5 billion
29/11/2008	200	40 000	N/A

3 Damage pattern

The majority of the existing buildings in the region have not been designed to withstand earthquakes. Most of them are typically unreinforced masonry low-rise buildings and have been exiguously designed. This structural type is ubiquitous all over the region. Heavy damages and collapses after earthquakes demonstrated the vulnerability of such unreinforced masonry buildings.

The buildings which have received the most severe damage in the recent earthquakes are mostly unreinforced stone, concrete block, and masonry buildings (see Fig. 6). Figure 6 shows some example buildings suffering such damage (Naseer et al., 2007; Khan and Khan, 2008). Performances of stone masonry schools, colleges, universities, old hospitals and official buildings were very bad during the earthquakes. Old age, poor construction and materials, and improper design were among the main factors for bad performance. Damages suffered by the masonry buildings could be explained by diagonal shear failure, combined in and outplane effects, flexure failure of pier, failure of building corners, separation of orthogonal walls, damages at walls and failure of external masonry and parapet walls (Amjad et al., 2007).

In the region, the construction application is based on experiments performed by contractors. In general, the stone masonry buildings have the light weight wooden roof trusses instead of a roof or floor diaphragm to brace the walls. Therefore, the walls are allowed to span horizontally to perpendicular walls that are less likely to withstand an earthquake. The spacing between walls is wider (Green, 2007; Tolles et al., 2000).

After the Kashmir earthquake in 2005, the Pakistani Earthquake Reconstruction-Rehabilitation Committee was established to organize and manage the construction of buildings in the damaged areas. The committee stated that structural designs of buildings must be approved by this committee for compliance with international codes. The Pakistan Engineering Council was established to revise and update the Pakistan Building Code. The committee also decided to adopt the 1997 edition of Uniform Building Code and to modify its provisions to make it compatible with Pakistan. The 2007

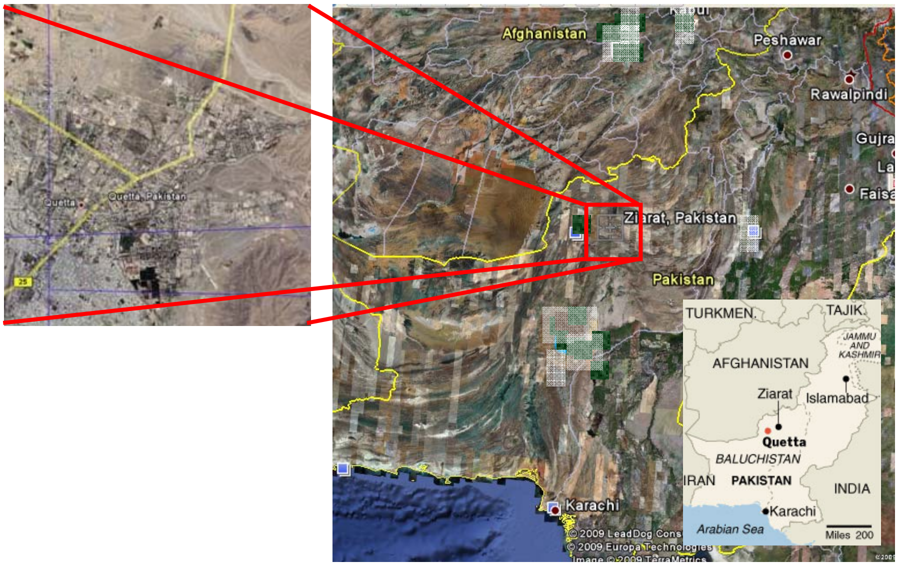


Fig. 2. Map of Pakistan and Location of Ziarat.

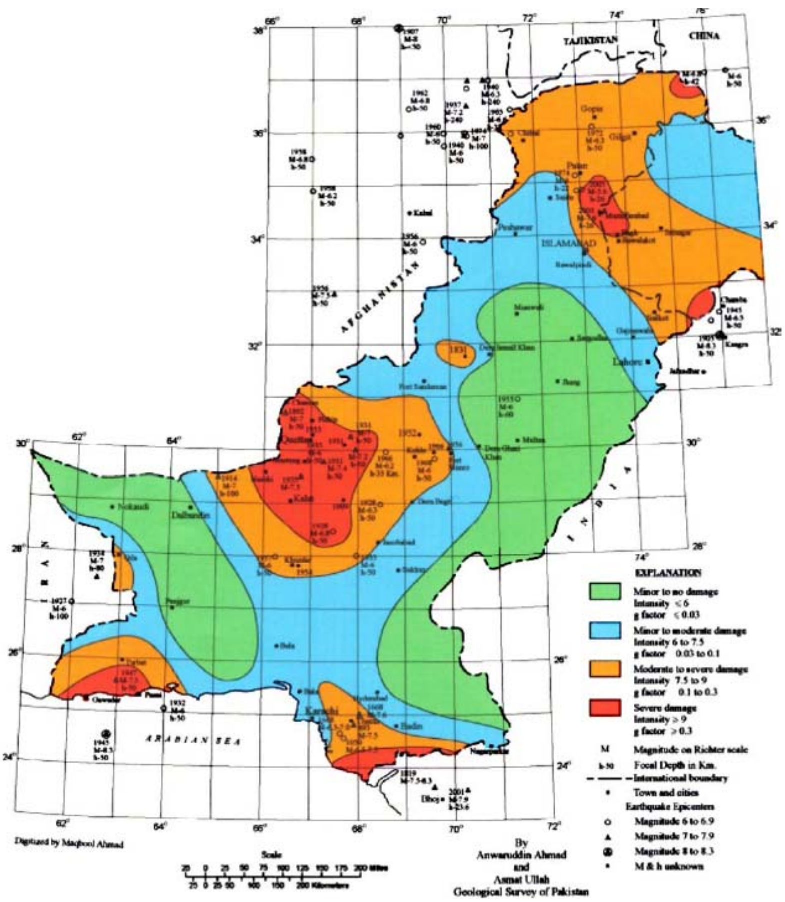


Fig. 3. Seismic Hazard Map of Pakistan (GSP, 2009).

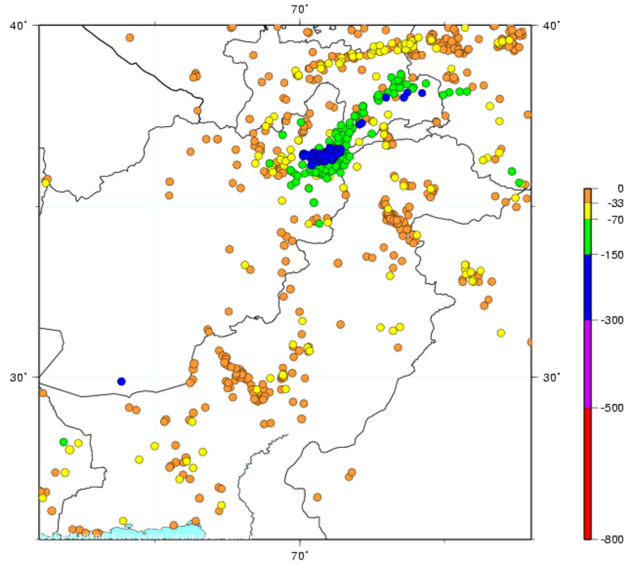


Fig. 4. Recent Earthquakes with the Magnitude ($M>5$) in Pakistan and its neighbourhood (USGS, 2009).

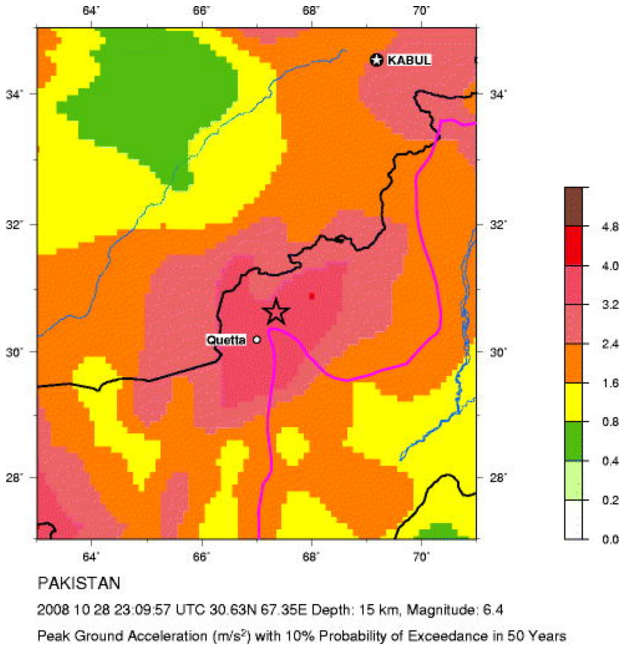


Fig. 5. Seismic Hazard Map for Pakistan (USGS, 2009).

edition of the Pakistan Building Code, which contains recommendations for detailed seismic design, is now in use in the entire country. The new code is mostly based on the 1997 edition of the Uniform Building Code.

4 Seismic assessment

There are two main seismic assessment approaches, namely deterministic and probabilistic. The deterministic based seismic assessment is comparatively simple and does not account for the uncertainties and probability of occurrence of an earthquake. For accurate deterministic seismic assessment, it is recommended that related long term ground motion data should be used in the analyses. However, this type of data is lacking or limited for most regions. A similar situation of lack of sufficient data also exists for this region (Lisa et al., 2005). For such cases, probabilistic based seismic assessments are more reliable. Probabilistic based assessment is denoted by the probability of seismic intensities exceeding a particular value within a specified time interval. In probabilistic based seismic assessment, the seismic vulnerability assessment is explained by a recurrence relationship, defining the cumulative number of events per year versus their magnitude. Analyses for different sites can be used to generate hazard curves which define seismicity of the region. In seismic assessment, seismic hazard assessment is used for the determination of the seismic ground motion of the region; seismic safety assessment is used for explaining the seismic vulnerability of the buildings.

4.1 Seismic hazard assessment

Seismic hazard assessment can be explained in two ways, pre- and post- evaluations. Pre-evaluation is done immediately following an earthquake. Rapid visual assessment is one of the important tools for a fast evaluation. The methodology provided in HAZUS is a post evaluation model that can be applied in regions that are at risk of earthquake disaster. For long-term evaluation, detailed post-earthquake evaluation is done to consider the structural damage. Long term evaluation includes nonlinear based performance evaluation analyses.

HAZUS methodology generates estimates of the consequences to a city or region due to a scenario earthquake. A scenario earthquake could be a specified earthquake with magnitude and location. The evaluation methodology consists of three basic steps (Korkmaz, 2009; Schneider and Schauer 2005).

1. Study region definition: definition of the region, selection of the application area, and selection of the appropriate data from the earthquakes.

2. Hazard characterization: definition of the earthquake hazard, hazard type and source, fault type, and earthquake location.

3. Damage and loss estimation: social and economic loss estimation, structural hazard estimation.

HAZUS, is a user friendly risk assessment model that can address the seismic hazard in the US. It was developed more

Fig. 6. Various damages after the earthquake (Naseer et al., 2007; Khan and Khan, 2008).

than 17 years ago in the US. It is a state-of-art decision support tool for assessing disasters. Capabilities in HAZUS include earthquake, flood and hurricane hazard characterization, building, essential facilities damage analysis, computing direct economic losses and secondary hazards (FEMA, 1999).

However, HAZUS is not ready for use in Pakistan and its neighbourhood. National and district boundaries and characterization of the earthquake data used in HAZUS are currently only available for the US. HAZUS can be used to provide a starting point for the development of a disaster risk assessment tool which could be used in Pakistan and its neighbourhood considering user requirements and data availability (FEMA, 1999).

4.2 Seismic safety assessment

In the study, seismic safety assessment was used for evaluation of buildings. Buildings were evaluated using nonlinear analyses. There are various different methodologies available in the literature for modelling and nonlinear analyses (Agbabian, 1984; Bruneau, 1994; Kim and White, 2004; Lam et al, 2003; Mistler et al, 2006; Moon et al, 2006; Priestly, 1985; Sucuoglu and Erberik, 1992; Vera et al, 2008; Yi et al, 2006).

Since the concern is unreinforced masonry buildings for the region, the evaluation of the seismic behaviour of unreinforced masonry buildings requires specific procedures. The masonry buildings' response to dynamical loads often differs substantially from those of ordinary buildings. In order to obtain a reliable estimation of the seismic risk, it is desirable to perform full dynamic analyses that describe the effective transmission and dissipation of the energy coming from the ground motion into the building (Pena et al., 2007).

Probabilistic seismic safety assessment for unreinforced masonry buildings is currently available from HAZUS, a seismic loss estimation framework developed by the Federal Emergency Management Agency (FEMA, 1999). HAZUS uses a systematic approach for probabilistic damage assessment of buildings, in which building response is characterized by building capacity curves and seismic hazard is represented by demand spectra for example, capacity spectrum method (Park, 2009).

Defining seismic risk is the first important step of probabilistic based assessments. Probabilistic based assessments consider probabilistic parameters and variables. FEMA proposes performance based procedures. The procedures do not consider probabilistic variables. In the processes, building performance and levels of damage to structural components are considered. Building performance or damage levels are specified as a function of the maximum drift the building sustains during an earthquake (FEMA, 2000; Park et al, 2009).

For unreinforced masonry buildings, three performance levels are defined in FEMA 356 as Collapse Prevention, Life Safety, and Immediate Occupancy. HAZUS defines four damage levels as Slight, Moderate, Extensive, and Complete Damages. The procedure does not consider probabilistic variables. HAZUS uses the maximum drift ratio as the damage measure following FEMA 356, and the threshold values of the maximum drift ratio corresponding to the limit states

Table 2. Material properties.

Material	Modulus of Elasticity (Mpa)	Poisson Ratio	Density kN/m^3
Regional Masonry Stone Material	26 000	0.2	25

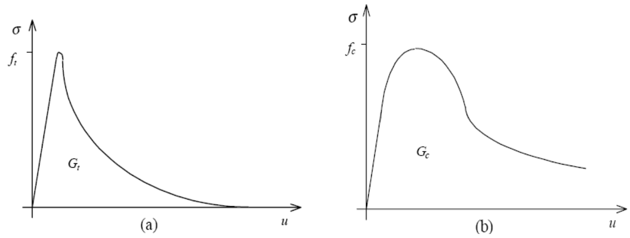


Fig. 7. Typical behaviour of quasi-fragile materials under uniaxial loading and definition of the fracture energy **(a)** tension loading **(b)** compression loading (Alvarenga, 2002).

are defined based on comprehensive review of past studies on seismic building damage (FEMA, 1999, 2000; Park, 2009).

In the study, seismic safety of unreinforced masonry low-rise buildings in Pakistan and its neighbourhood was investigated. Therefore, first, nonlinear time history analyses then probabilistic based assessment analyses were conducted for representative buildings. In the following section, modelling and analysis of the representative building are explained.

5 Seismic safety assessment of unreinforced masonry low-rise buildings in Pakistan and its neighbourhood

In the study, structural performances of unreinforced masonry low-rise buildings in Pakistan and its neighbourhood were estimated through nonlinear time history analysis. Therefore, four different representative buildings were modelled and analyzed.

5.1 Modelling representative buildings

When modelling representative buildings two critical modelling issues should be considered: (a) representation of the real structural behaviour and (b) non-linear material relationships. Masonry consists of mainly unit element and mortar. Most common unit elements are brick and stone. Mortar is used for connecting the units. Compressive, tensile and shear strength, durability, water absorption coefficient and thermal expansion affect the load bearing capacity of masonry. Masonry buildings include blocks connected by mortar joints that are geometrically complex and reflected in the computational effort needed. Modelling of joints is specifically important, since the sliding at joint level often starts the crack propagation. Mortar joints in masonry buildings cause the masonry to be anisotropic. Two different approaches have been adopted to model such anisotropy: the "micro-model" or "two-material approach" and the macro-model. In both models, the discretization follows the actual geometry of both the blocks and mortar joints, adopting different constitutive models for the two components.

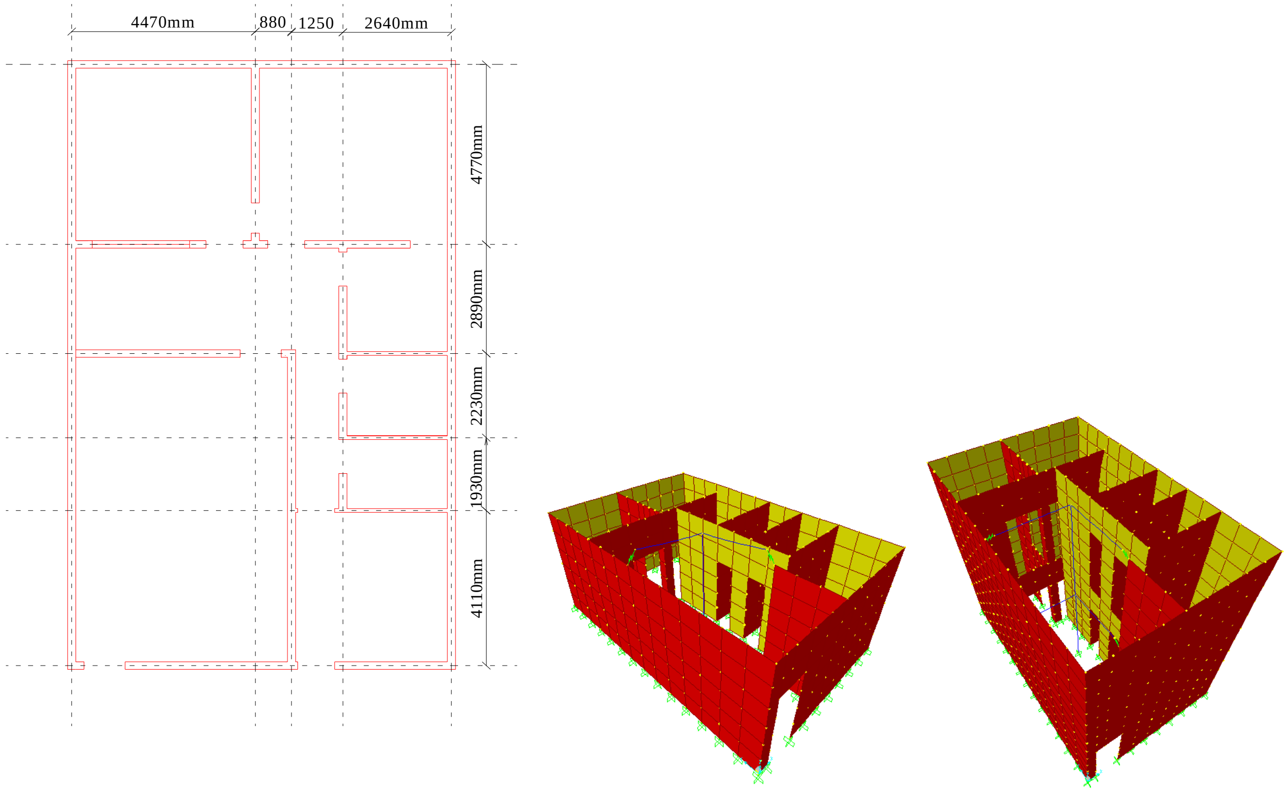
Strength of stone masonry depends on the material properties and bond materials used. The stone is massive and stiff. Type and thickness of mortar is more effective on the compressive strength of stone masonry than stone units. The strength of stone does not have much effect on stone masonry. The joint behaviour of unit and mortar determines the strength of stone masonry. If the mortar strength is weaker than units, masonry strength primarily depends on the mortar strength. The shear strength of the stone masonry is approximately 25% of the compressive strength (Unay, 2002; Lourenco, 1998).

To demonstrate the construction type and determine a path for seismic safety assessment for the region, four different representative buildings were modelled similar to the real examples. The existing building types in the region were adopted from the literature and real examples (Naseer et al., 2007). Models shown in Figs. 8 and 9 were nonlinearly investigated via nonlinear time history analyses (Wilson and Habibullah, 1998). As depicted in Figs. 8 and 9, the plan areas of typical masonry buildings in the region are close to these values. Material properties are given in Table 2. Material properties are defined according to the stone masonry material which is commonly used for construction in the region. Masonry constructions subjected to seismic events can present damping ratios around 8%–10% (Rivera, 2008). In the present study, damping ratio was taken as 8%.

5.2 Seismic safety assessment of representative buildings

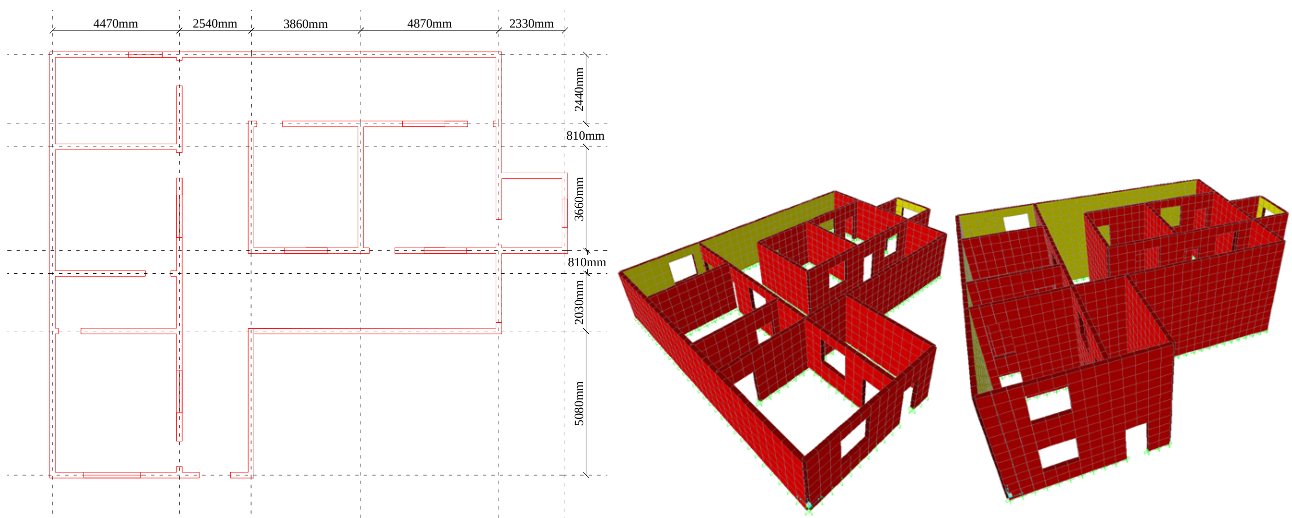
To obtain a set of performance estimates for representative buildings, a series of nonlinear time history analyses were conducted (Wilson and Habibullah, 1998). An extraordinarily important step for application of time history analysis is the selection of the earthquakes. For each model, 60 different ground motion data were selected. Since, local data is not available for the region, the earthquake data was chosen from all over the world. Non-linear dynamic analyses were performed considering tensile strength, tensile fracture energy and damping. In total, 240 time history analyses were conducted for evaluation. Table 3 lists the peak ground accelerations (PGA) and peak ground velocity (PGV) of the records. The time history results are given in Figs. 10 and 11.

After time history analyses, probabilistic evaluations for drift levels were made to provide performance likelihoods during design-basis earthquakes expected in the future. Drift demand versus exceedence probability curves were



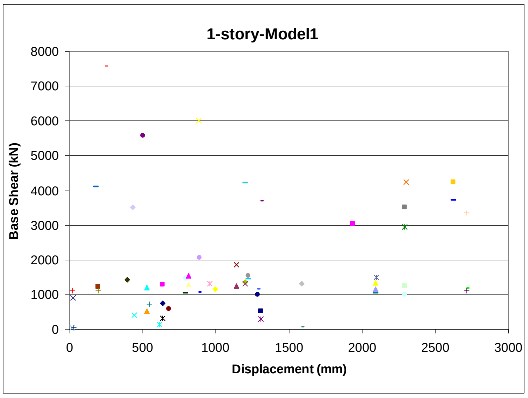
(a) Plan of model 1 (b) One story model 1 (c) Two story model 1

Fig. 8. Plan and 3-D view for one and two-story of Model 1.

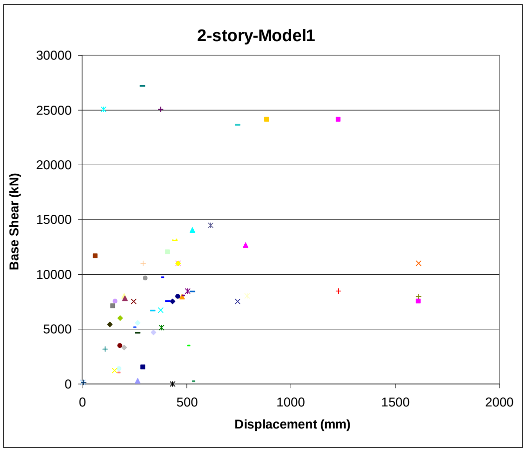


(a) Plan of model 2 (b) One story model 2 (c) Two story model 2

Fig. 9. Plan and 3-D view for one and two-story of Model 2.



(a) One story Model 1



(b) Two story Model 1

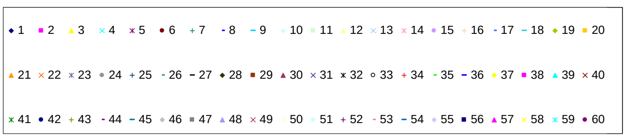
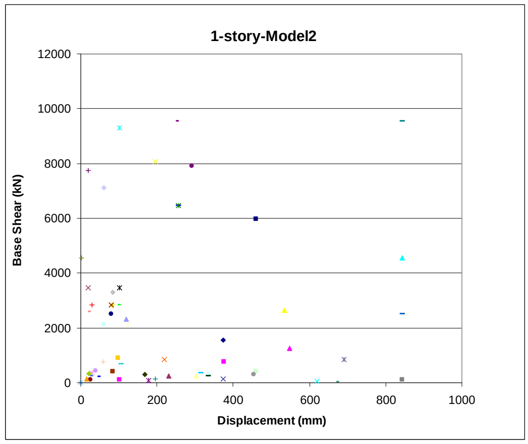
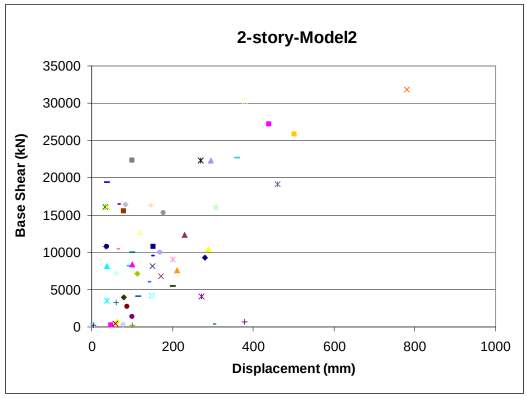


Fig. 10. Time history analysis results for Model 1.



(a) One story Model 2



(b) Two story Model 2

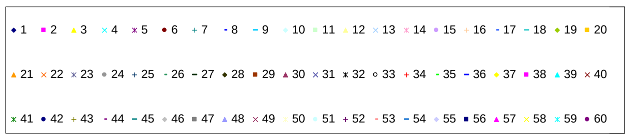


Fig. 11. Time history analysis results for Model 2.

developed based on analysis results and simplified probabilistic modelling. For each building, the mean and standard deviation values of these maximum drifts were assumed as parameters of normal distribution functions. According to Cornell et al., a log-normal distribution for the statistical description of the building response would be a reasonable assumption (Cornell et al., 2002). By using the distribution functions, probability of exceeding the different drift levels was computed for each model (Park et al., 2009). These distribution functions gave the probabilistic drift demand curves (see Fig. 12). Using these drift demand curves, probability of exceeding different displacement capacity thresholds (displacement limits) chosen from displacement capacity curves for each model were estimated.

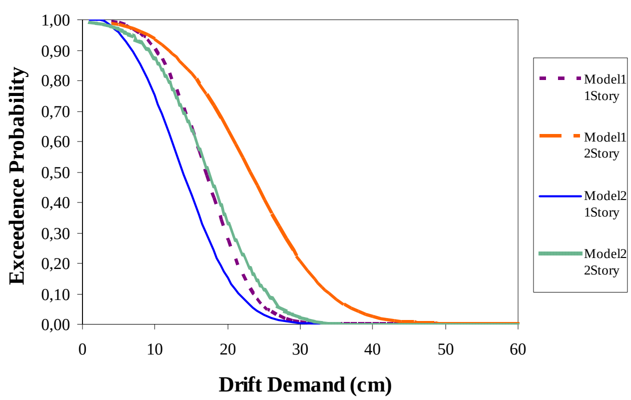


Fig. 12. Probabilistic seismic safety assessment.

Table 3. Earthquake data.

#	Name	Date	Magnitude (M_w)	Code	PGV (cm/s)	PGA (g)	Distance (km)
1	Anza (Horse Cany)	25/02/1980	4.9	AZF315	2.6	0.066	12.1
2	Morgan Hill	24/04/1984	6.2	G01320	2.9	0.098	16.2
3	Coyote Lake	06/08/1979	5.7	G01320	8.3	0.132	9.3
4	Landers	28/06/1992	7.3	GRN180	14.1	0.041	141.6
5	Landers	28/06/1992	7.3	ABY090	20	0.146	69.2
6	Landers	28/06/1992	7.3	SIL000	3.8	0.05	51.7
7	Landers	28/06/1992	7.3	29P000	3.7	0.08	42.2
8	Loma Prieta_	18/10/1989	6.9	G01090	33.9	0.473	11.2
9	Loma Prieta	18/10/1989	6.9	SGI360	8.4	0.06	30.6
10	Loma Prieta_	18/10/1989	6.9	MCH000	3.5	0.073	44.8
11	Loma Prieta_	18/10/1989	6.9	PTB297	12.9	0.072	78.3
12	Lytle Creek_	12/09/1970	5.9	CSM095	1.8	0.071	88.6
13	N. Palm Springs	08/07/1986	6.0	AZF225	5.8	0.099	20.6
14	N. Palm Springs_	08/07/1986	6.0	ARM360	3.4	0.129	46.7
15	N. Palm Springs_	08/07/1986	6.0	H02090	1.8	0.093	45.6
16	N. Palm Springs_	08/07/1986	6.0	H02000	1.9	0.07	57.6
17	Whittier Narrows	01/10/1987	5.3	MTW000	40	0.123	20.4
18	Anza (Horse Cany)	25/02/1980	4.9	AZF225	3.3	0.065	12.1
19	Anza (Horse Cany)	25/02/1980	4.9	PTF135	5.1	0.131	13
20	Anza (Horse Cany)	25/02/1980	4.9	TVY135	1.7	0.081	5.8
21	Coyote Lake	06/08/1980	5.7	G01-UP	2.5	0.072	9.3
22	Düzce	12/11/1999	7.1	1060-E	5.3	0.053	30.2
23	Düzce	12/11/1999	7.1	1060-N	11	0.028	30.2
24	Hollister	28/11/1974	5.2	G01247	4.0	0.132	12.3
25	Kocaeli	17/8/1999	7.4	GBZ000	50.3	0.244	17
26	Kocaeli	17/8/1999	7.4	GBZ270	30	0.137	17
27	Cape Mendocino	25/4/1992	7.1	CPM-UP	63	0.754	8.5
28	Loma Prieta_	18/10/1989	6.9	RIN090	10.4	0.092	79.7
29	N. Palm Springs_	08/07/1986	6.0	WWT180	34.7	0.492	7.3
30	Whittier Narrows	01/10/1987	5.3	MTW090	35	0.036	21.2
31	Parkfield	28/06/1966	5.6	C12320	6.8	0.0633	14.7
32	Morgan Hill	24/04/1984	6.2	GIL067	3.6	0.1144	16.2
33	Kocaeli	17/08/1999	7.4	ARC000	17.7	0.2188	17
34	Morgan Hill	24/04/1984	6.2	G06090	36.7	0.2920	11.8
35	Coyota Lake	06/08/1979	5.8	G06230	49.2	0.4339	3.1
36	Northridge	17/01/1994	6.7	ORR090	52.1	0.5683	22.6
37	Loma Prieta	18/10/1989	7.1	CLS000	55.2	0.6437	5.1
38	Kobe	16/01/1995	6.9	KJM000	81.3	0.8213	6.9
39	Santa Barbara_	13/08/1978	7.2	SBA222	16.3	0.203	14.0
40	Livemor	27/01/1980	7.4	LMO355	9.8	0.252	8.0
41	N. Palm Springs	08/07/1986	6.0	NPS300	33.8	0.694	8.2
42	Loma Prieta	18/10/89	6.9	STG000	41.2	0.512	13.0
43	Northridge	17/01/1994	6.7	TPF000	17.6	0.364	37.9
44	San Fernando	02/09/1971	6.6	ORR021	15.6	0.324	24.9
45	Whitter Narrow	10/01/1987	6.0	ALH180	22	0.333	13.2
46	Kocaeli	17/08/1999	7.4	SKR090	79.5	0.376	3.1
47	Victoria, Mexica	09/06/1980	6.1	CPE045	31.6	0.62	34.8
48	Anza (Horse Cany)	25/02/1980	4.9	BAR225	2.6	0.047	40.6
49	Anza (Horse Cany)	25/02/1980	4.9	RDA045	6.7	0.097	19.6
50	Borrego Mtn	09/04/1968	6.8	PAS270	4.7	0.090	203.0
51	Coyote Lake	06/08/1979	5.8	SJ3337	7.6	0.124	17.2
52	Coyote Lake	06/08/1979	5.8	SJ5337	7.4	0.114	17.2
53	Imperial Valley	15/10/1979	7.0	CPEDWN	6.8	0.116	8.3
54	Imperial Valley	15/10/1979	7.0	PTS315	16.1	0.204	14.2
55	Hollistr	28/11/1974	5.2	D-SG3295	9.3	0.339	14.9
56	Cape Mendocino	25/04/1992	7.1	EUR090	28.3	0.178	44.6
57	Cape Mendocino	25/04/1992	7.1	FOR000	30	0.116	23.6
58	Kern County	21/07/1952	7.4	PAS180	5.6	0.045	127.0
59	Kern County	21/07/1952	7.4	SBA132	15.5	0.127	87.0
60	Loma Prieta	18/10/1989	6.9	A09137	15.6	0.113	46.9

6 Conclusions

The present study investigates the seismic safety of common unreinforced masonry low-rise buildings in Pakistan and its neighbourhood, using different assessment and estimation approaches. Probabilistic based seismic safety assessment is processing of a combination of nonlinear analysis and probabilistic approach. Time history analyses were conducted with 60 different earthquake data (Table 3). According to time history analysis results, four different representative building models were evaluated. The analysis results show that there are significant amounts of displacements at the unreinforced masonry low-rise buildings. Analysis results indicate quite different responses of the representative buildings to earthquakes. The non-linear time history analyses indicated that masonry buildings are susceptible to seismic damage that has higher displacements.

The results of the analyses are given in the figures. According to analyses results, the studied types of unreinforced masonry low-rise buildings in Pakistan present higher displacements and shear forces that can cause damage or collapse. By using analysis results, seismic vulnerability of unreinforced masonry low-rise buildings in the region was investigated. Seismic safety of the investigated unreinforced masonry low-rise buildings is questionable.

There are many lessons learned from the recent earthquakes. Based on these lessons, in the short and long term, measurements in seismic prone areas in Pakistan and its neighbourhood are essential. In conclusion, a detailed seismic safety assessment for all unreinforced masonry low-rise building types in the region is very important. The next step could be definition of strengthening of unreinforced masonry buildings via simplified and fast methodologies.

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