Displacement-Based Seismic Risk Assessment of Stone Masonry Buildings of Pakistan

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Abstract

The paper presents the calibration, using experimental and numerical investigations, of a nonlinear static analytical displacement-based method for seismic risk assessment of stone masonry buildings of Pakistan. The displacement-based method uses the mechanical models: defined completely by secant vibration period, drift limit states and viscous damping of structural material, and overdamped displacement spectrum to assess the seismic performance of structures and quantify regional seismic risk. A one-third scaled model, representing the existing construction practices, single storey one room rubble stone masonry buildings is tested on shake table using real accelerograms and incremental excitation to reproduce the capacity curve of the considered building. The experimental data is analyzed to obtain the lateral strength, drift limits, and damage scale of rubble stone masonry. Furthermore, the experimental data is used to develop prototype 2D structural models for nonlinear dynamic time history analysis of stone masonry buildings using equivalent frame method. Two storey 2D structural models are designed, respecting the considered material properties, and analyzed using real accelerograms to derive equivalent static single degree of freedom systems and their corresponding secant vibration period for the considered building typologies and record-to-record variability in the capacity parameters. Controlled Monte Carlo simulation is used to generate random populations of regional building stock, taking into account the uncertainties also in the geometrical and mechanical properties of the structures, to develop displacement-based fragility functions which can be used to derive damage probability matrices and socio-economic losses in the region for public awareness and community earthquake preparedness planning in order to mitigate the future expected regional risk.

Keywords: displacement-based; nonlinear static; seismic risk assessment; fragility functions; damage scale; stone masonry; Pakistan.

1. Introduction

Stone masonry buildings constitute a substantial portion of the total building stock of northern areas of Pakistan. Based on the combination of different systems of walls, roof and floors, a variety of stone masonry buildings exist in northern parts of country. However, the main construction techniques and structural features of these buildings are fairly uniform throughout the region. Two wythes random rubble stone masonry walls in dry, mud mortar or cement mortar with flat earthen, pitched G.I. sheet roof or reinforced concrete (rc) slab is the most common construction type. Recent practices make use of throw stones and vertical steel bars, 1.2m apart, to improve the performance which seems not to be dramatically different than the ordinary wall configuration. Nevertheless, the findings herein is conservative for these new systems.

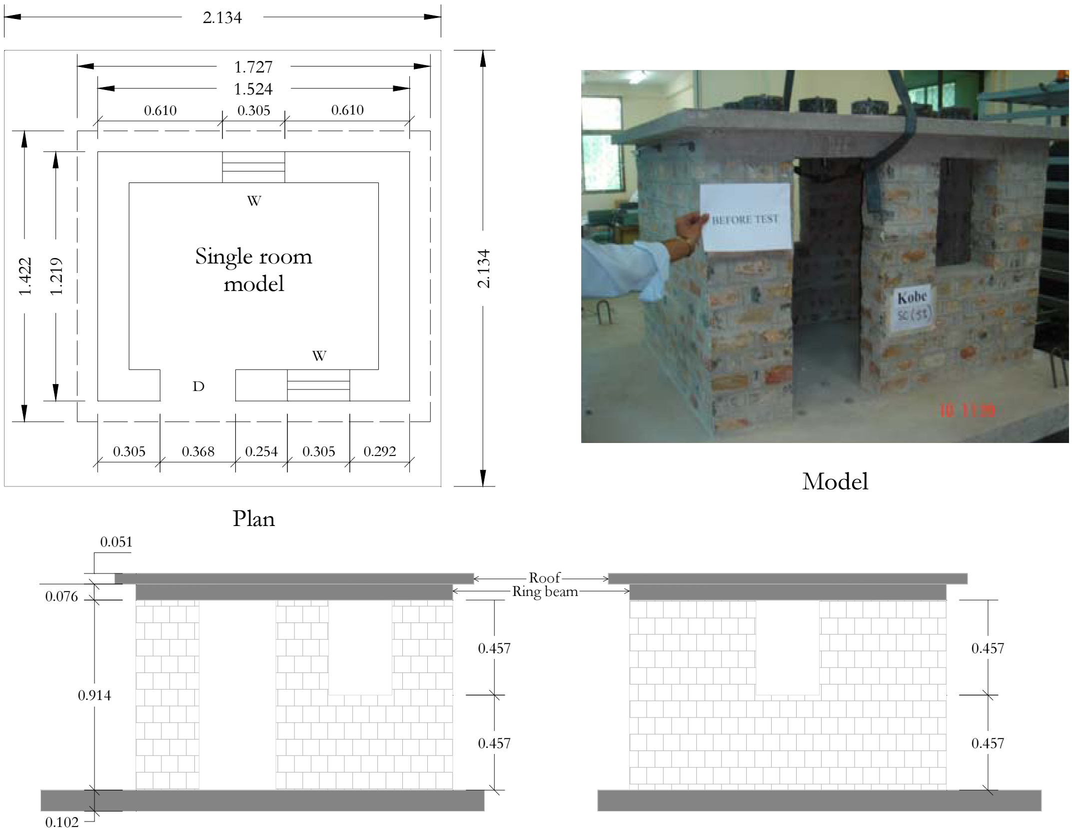
These building systems have shown very poor performance in past earthquakes and lead to huge losses of life and economy. Collapse of such structures featured prominently in the 2005 Kashmir earthquake [1]. Thus, the present paper assess the seismic performance of the considered building system using experimental and numerical investigation. Analytical methodology is developed for future applications in the vulnerability assessment and earthquake loss estimation of such buildings on regional scale for earthquake preparedness and risk mitigation in the region.

2. Experimental test on rubble stone masonry

2.1 Model description and test setup

Most of the mechanics based assessment approaches need the basic material properties of masonry to assess the capacity of masonry structural systems. The material properties; compressive strength, elastic moduli, tensile strength, etc, of rubble stone masonry cannot be obtained reliably at the section level due to the difficulties in performing tests on masonry prism, reproducing the true replica of the filed condition and huge scatter in the observed behaviour. Which is due to the fact that when stone-to-stone contact is found during the compression tests very huge value of compressive strength is achieved which get minimal value when stone-to-void possibility is found. Huge uncertainties in the material properties at the section level make the global response less reliable. Thus, a full reduced scaled model is tested to obtain the global capacity of the considered building typology.

One third scaled model of single storey and single room is designed using the similitude principles, see Fig. 1 for the structural detailing of the tested model. The model is constructed of half dressed stone masonry work in cement mortar with rc slab and ring beam which represents most of the existing building stock, public and private, in the region. The model is tested in the weaker direction on a single degree of freedom (SDOF) shake table at the Earthquake Engineering Center of Peshawar using real accelerogram, Kobe 1995, and incremental excitation with linear scaling until the complete collapse of the model. The model is instrumented with accelerometers and displacement transducers at the base of the model and top of the roof.


Plan

Model

Front Elevation

Back Elevation

Fig. 1: Details of the Model Tested on Shake Table

2.2 Observed response of the model

The model is subjected to 5% scaled Kobe record with linear amplitude scaling until the total collapse. The model is inspected for the observed damage after every run. The floor acceleration and displacements, on each in-plane walls and at the base of the model, are recorded and processed for noise removal for each excitation. The floor acceleration is normalized by the seismic mass of the structural model in order to obtain the base shear for the corresponding equivalent SDOF system. Also, the average floor displacement demand is obtained which is corrected with the base displacement demand in order to obtain the relative displacement demand on the system which is normalized by the model height to compute the corresponding drift demand on the system. Table 1 shows the capacity parameters and the observed damages at different performance levels of the prototype of tested stone masonry model.

Table 1: Capacity Parameters and Damage States of the Prototype System

Damage Level	Equivalent Base Shear (m/sec^2)	Drift Demand (%)	Damage Description
Minor (D1)	1.67	0.05	Separation of reinforced concrete slab and ring beams from walls
Moderate (D2)	3.92	0.48	Crack initiation in the masonry in-plane walls and around the openings
Major (D3)	5.00	1.49	Widening of cracks and falling of stones from the out-of-plane walls
Collapse (D4)	1.47	2.65	Complete collapse of the structural model.

3. Displacement-based seismic risk assessment of structures

The calibration of a nonlinear static analytical mechanics-based fully probabilistic method, presented herein, is performed for the seismic vulnerability and risk assessment of stone masonry building stock on regional scale. The displacement-based method is originally proposed by [2] for rc buildings and developed for brick masonry buildings of Pakistan by [3] and consequently further developed for rubble stone masonry buildings of Pakistan herein. The method is capable of incorporating sources of expected uncertainties in the seismic demand and structural capacity explicitly in contrary to the existing conventional procedures (e.g. [4] among others).

3.1 Nonlinear static SDOF systems, mechanical model, for stone masonry buildings

The seismic response of stone masonry buildings with reinforced concrete slab and ring beams is mainly governed by the global mechanism and shear response of the in-plane walls with limited energy dissipation capabilities and ductility level. An equivalent SDOF system is used to simulate the nonlinear response of an actual building in terms of displacement capacity at different performance levels, damage states, see Fig. 2 for an SDOF idealization of stone masonry buildings.

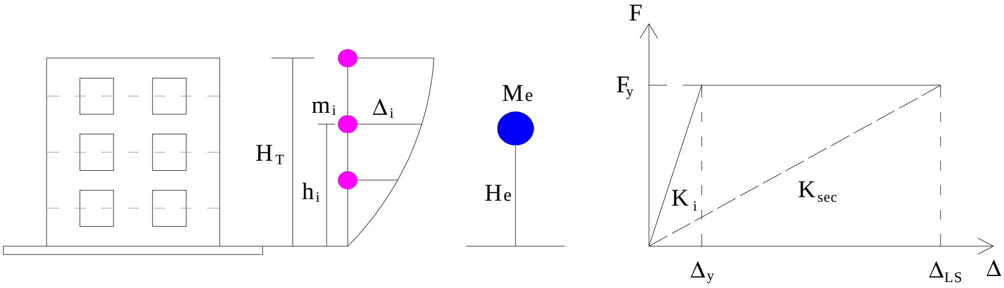


Fig. 2: Nonlinear Static SDOF Idealization of Stone Masonry Buildings

In this figure, H_T represents the total building height; h_i represents the i^{th} floor height, Δ_i represents the lateral displacement and m_i represents the i^{th} floor mass for a given deformed shape of the building; Me and He represent the mass and height of the equivalent SDOF system; Δ_y and Δ_{LS}

represent the equivalent yield and ultimate limit state displacement that represents the displacement capacity of the actual building at the center of seismic force for a specified deformed shape; K_i represents the initial pre-yield stiffness; F_y represents the yielding force; K_{sec} represents the secant stiffness. For seismic assessment, the static SDOF system is completely defined by secant vibration period, limit state displacement capacity and energy dissipation characteristics of buildings represented as viscous damping.

3.2 Derivation of damage probability matrices (DPM) and earthquake loss assessment

Controlled Monte Carlo simulation is used to generate random buildings with different geometrical and material properties representing regional building stock; the variability of each property being defined a priori using probabilistic distribution. Once the population is generated, the limit state displacement capacities, secant period and viscous damping of the buildings are computed using calibrated structure-specific empirical models and pre-defined damage grades. For a given earthquake, each of the building from the generation is analyzed for the capacity-demand check at secant vibration periods using overdamped displacement spectrum. The number of buildings having capacity less than the demand divided by the total number of generated buildings gives an estimate of the limit states probability of exceedance (Pfj). The number of buildings in a given damage state, DPM, is obtained as follows: undamaged (D0)=1-Pf_1; minor (D1)=Pf_1-Pf_2; moderate (D2)=Pf_3-Pf_2; major (D3)=Pf_4-Pf_3; collapse (D4)=Pf_4-Pf_3, which are used to estimate the socio-economic losses of earthquakes and develop regional risk maps [3].

4. Derivation of mechanical models for stone masonry buildings

4.1 Characteristics of the case study buildings

Stone masonry buildings are practiced abundantly in the northern areas, urban and rural, of Pakistan due to the large local availability of stone material and low cost of labour in construction. Stone masonry in cement mortar with earthen roof, G.I. sheet or rc roof are the common residential building construction practice for single storey, in rural areas, and two storey with rc floors, in urban areas. Stone masonry in cement mortar contribute 50% in overall to the total building stock in the northern areas (*www.erra.gov.pk*). The most prevailing building dimensions ranges from 8m×5m to 15m×5m with typical wall density, the ratio of the cross sectional area of the in-plane walls to the total floor area, ranging from 10% to 15% [4]. These buildings have 300 to 500 mm thick load bearing walls with rubble masonry; having 130 to 150 mm thick rammed earthen roof, G.I. sheet or rc slab; inter storey height of 2.0 m to 3.0 m. These buildings are provided with a ring beams right above the walls, approximately 150 to 230 mm deep and width equal to the wall thickness. The building rests on shallow strip type footing, with stepped stone work overlain compacted earth (sub grade). The load bearing walls, 70% of the total wall length, are perforated by doors, with typical dimensions of 1m×2.13m, and windows, with typical dimensions of 1m×1.22m to 1.83m×1.22m. The primary seismic resistance mechanism, as observed in the dynamic test is in-plane global mechanism with diagonal shear cracks in the short brick piers. However, the ultimate mechanism is governed by the combined in-plane and out-of-plane failure of the masonry walls.

4.2 Prototype 2D cases study structural models

4.2.1 Design of Prototype buildings

Due to the unavailability of basic material properties for rubble stone masonry walls, it is not straight forward to develop numerical tools for future applications. Thus, the present study considered the simplified equivalent frame approach, *SD-SAM* proposed by [6] for nonlinear static and dynamic time history analysis of masonry buildings, and developed herein for rubble stone masonry using indirect approach.

In the first step, a generic prototype of equivalent frame is generated for the tested model and designed, respecting the geometric detailing and loading conditions, with the available empirical shear strength models for masonry walls. The possible failure mechanisms considered for unreinforced masonry piers are the flexure or rocking and shear failure using the following strength models:

$$V_f = \frac{p.D^2 t}{2\varphi H_p}\left(1 - \frac{p}{kf_u}\right) \tag{1}$$

$$V_d = \frac{f_{tu} Dt}{b}\left(1 + \frac{p}{f_{tu}}\right)^{1/2} \tag{2}$$

$$V_S = \frac{Dt(1.5c + \mu p)}{\left(1 + 3\frac{c\varphi H_p}{pD}\right)} \tag{3}$$

where V_f represents the ultimate strength for flexure/rocking failure; D and t represent the length and thickness of the pier; p=P/(D.t) represents the mean vertical stress due to axial load P; H_P represents the total height of the pier; ψ is 1.0 for a cantilever pier and 0.5 for a pier fixed-fixed boundary conditions; f_u represents the compressive strength of the masonry; k represents the coefficient used to idealize the stress distribution at the compressed toe of the pier, taken as 0.85; V_d is the diagonal shear strength; f_{tu} represents the diagonal tensile strength; b=1 for $H_P/D \leq 1$, $b=H_P/D$ for $1<H_P/D<1.5$ and b=1.5 for $1.5 \leq H_P/D$; V_S represents the sliding shear strength; μ, assumed as 0.4, and c, taken as $0.4f_{tu}$, represent the coefficient of friction and cohesion of masonry as global strength parameters. The strength computed is reduced by 10% in order to respect the energy balance criterion . In the second step, the tensile strength, f_{tu}, of masonry is selected, fixing the compressive strength to 4 Mpa, to achieve the equivalent yield strength, 4.50 m/sec^2, of the corresponding prototype systems. In the next step, the Young modulus, E, and shear modulus, G=0.3E, are selected to achieve the corresponding drift limits at yield and the 1st modal period of the system. The present assumption for compressive strength and shear modulus is typical for such type of masonry system [7]. The capacity curve of the designed frame is obtained using *SD-SAM*, employed in OpenSees [8], with displacement controlled pushover analysis which is compared with the observed capacity of the tested prototype stone masonry building and is found reasonable in terms of yield stiffness and equivalent bi-linearised strength, respecting the energy balance criterion [9].

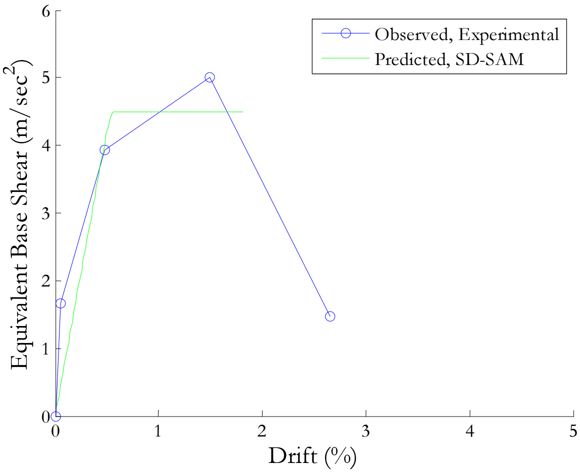
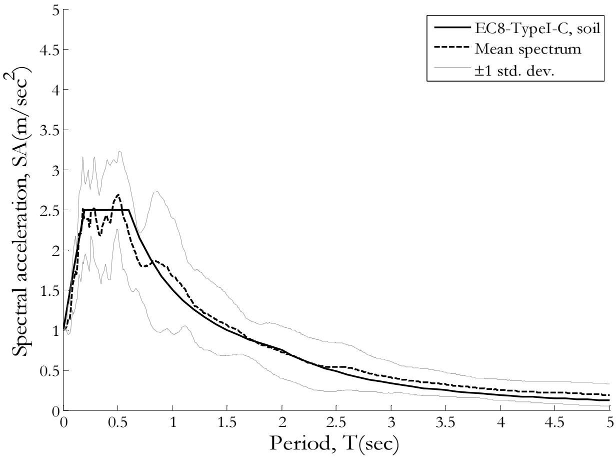


Fig. 3: Comparison of the Observed and Predicted Response of Tested Stone Masonry building

4.2.2 Dynamic analysis of cases study structural models

The present study considered 125 cases study 2D two storey structural models; 5 cases for different floor area, 5 cases for different wall density and 5 cases for different material properties, designed with the material properties obtained in the previous section. The frame elements are assigned with bi-linear Takeda type rule with beta of unloading stiffness considered as 0.6 as proposed by [6] for masonry walls. The cases study models then are analysed dynamically using nonlinear time history analysis (NLTHA) with 10 real accelerograms extracted from the PEER NGA data base with the mean spectrum compatible to EC8 Type I C-soil spectrum [10], see Fig. 4 for the details of the accelerograms used in the present study. The accelerograms are linearly scaled in order to observe the post-yield response of the models which is used then to derive static SDOF system for the considered building typology and retrieve the dynamic characteristics of the considered buildings.



Record No.	Date	Event	Station/Component	Moment Magnitude (Mw)	Distance (km)	Soil Type (NEHRP)	Duration (sec)	PGA (g)
1	4/25/1992	Cape Mendocino	Fortuna - Fortuna Blvd	7.1	23.6	C	44.00	0.116
2	6/28/1992	Landers	Desert Hot Springs	7.3	23.2	C	50.00	0.154
3	6/28/1992	Landers	Yermo Fire Station	7.3	24.9	D	44.00	0.152
4	10/18/1989	Loma Prieta	Hollister Diff. Array	6.9	25.8	D	39.64	0.279
5	1/17/1994	Northridge	Beverly Hills	6.7	19.6	C	29.99	0.416
6	1/17/1994	Northridge	Canoga Park - Topanga Can	6.7	15.8	D	24.99	0.356
7	1/17/1994	Northridge	LA - Hollywood Stor FF	6.7	25.5	D	40.00	0.231
8	1/17/1994	Northridge	Sunland - Mt Gleason Ave	6.7	17.7	C	29.99	0.157
9	11/24/1987	Superstitn Hills(B)	El Centro Imp. Co. Cent	6.7	13.9	D	40.00	0.258
10	11/24/1987	Superstitn Hills(B)	Plaster City	6.7	21.0	D	22.23	0.186

Fig. 4: Mean of the acceleration spectra considered for NLTHA (left) and details of the accelerograms used (right)

The scope of the dynamic analysis is to compute the equivalent base shear and equivalent displacement demand at the yield limit state of the masonry walls on the ground floor using the proposed SDOF derivation of [6] and compute the vibration period of the building:

$$T_y = 2\pi\sqrt{\frac{\Delta_{eq}}{VB_{eq}}} \tag{4}$$

where Δ_{eq} represents the equivalent displacement and VBeq represents the equivalent base shear obtained from the normalisation of the floor displacements and base shear over the deformed shape and seismic mass participation of the building [6]. The vibration periods obtained for the considered cases study buildings are used to develop the empirical period model (5) for future applications:

$$T_y = a\exp(\pm\varepsilon\beta)H^b \tag{5}$$

where H represents the total height of the building; a and b are the coefficients obtained through the regression analysis; β represents the variability in the period computation. The period coefficient with b set to 0.75 obtained for 25 structural models with record-to-record variability and mean material properties is shown in Fig. 5 (left).

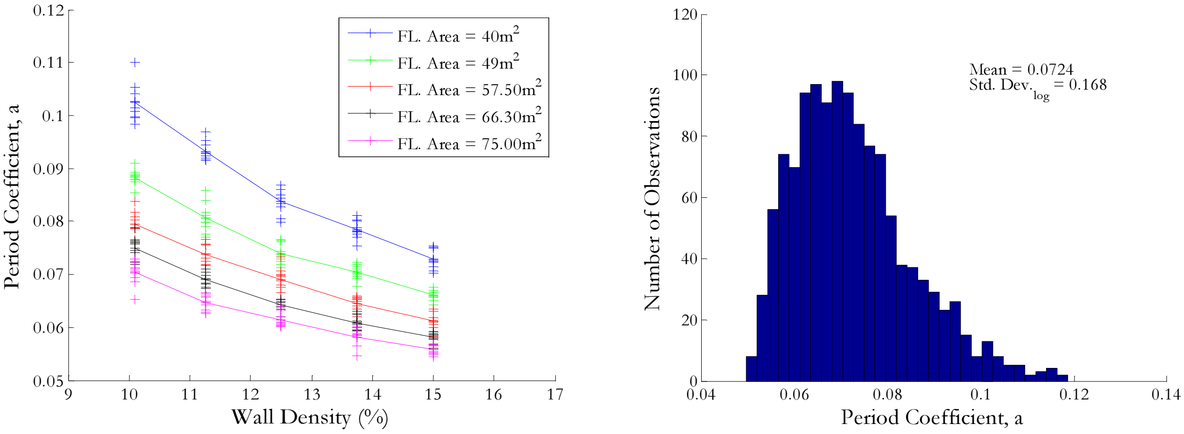


Fig. 5: Period Coefficient for Different Wall Density and Floor Areas (left) and for all Cases Study Structural Models (right)

For a given structural system, the period reduces with increasing wall density, for a given floor area, and increasing floor area. The record-to-record variability reduces with increasing wall density. The period coefficient obtained for all the cases study structural models, 1250 cases, is shown in Fig. 5 (right) which takes in to account the geometric (floor area and wall density) and material uncertainties besides the variability introduced by earthquake loading.

Additionally, the derived SDOF system properties are used to develop the equivalent displacement capacity model, formulated by [11]:

$$\Delta_{LS} = \theta_y k_1 H + (\theta_{LS} - \theta_y) k_2 h_s \tag{6}$$

where Δ_{LS} represents the limit state displacement capacity; H represents total height of the building; θ_y and θ_{LS} represent drift capacity at yield and post-yield limit states; h_s represents interstorey height of the building; k_1 and k_2 are the coefficients to obtain the equivalent height of the structural system, which are used herein to take into account the record-to-record variability in the displacement capacity as well and make the model (6) to treat different uncertainties explicitly. In the first step, static analyses are performed to obtain the crack and yield limit state drift values for all the cases study models using the shear strength and crack/yield stiffness of masonry walls respectively. A mean value of 0.24% is obtained for crack limit state with logarithmic standard deviation of 0.16, min 0.16% and max 0.33%. Similarly, for yield limit states, a mean value of 0.35% is obtained with logarithmic standard deviation of 0.15, min 0.24% and max 0.50%. These values are less than the experimental results due to the fact that the tested model has wall density of 12.84%. The computed drift values take into account the geometric variability, 125 cases, for rather broader range of wall density and floor areas. Using the limit states ductility obtained in the experiment, a mean value of 0.96% and 1.27% are obtained for the major and collapse limit states respectively. In the next step, the equivalent displacement obtained through NLTHA at the yield limit state are divided by the corresponding height of the structural model and wall drift limit in order to obtain k_1, for which a mean value of 0.72 is obtained with logarithmic standard deviation of 0.22. Similarly, the displacement capacity at post-yield limit state is used to obtain k_2, mean value of 0.8068.

5. Derivation of displacement-based fragility functions

The state-of-the-art and conceptual procedure proposed by [12] for the derivation of analytical fragility functions is used herein to derive displacement-based fragility functions for stone masonry buildings of Pakistan. In the first step, Controlled Monte Carlo simulation is used to generate thousands of SDOF system with different limit states mechanical properties using the developed models for periods and displacement capacity, respecting the regional variability in the geometric and material properties as well as the variability introduced by earthquake loading. In second step, linear displacement response spectrum are generated which are used to obtain the limit states probability of exceedance for the considered building typologies. Also, each of the generated spectrum is analyzed with the median mechanical properties to obtain the inelastic displacement demand on the system. For a given spectrum, the number of buildings exceeding different limit states is plotted against the displacement demand on that system in order to derive analytical fragility functions, see Fig. 6.

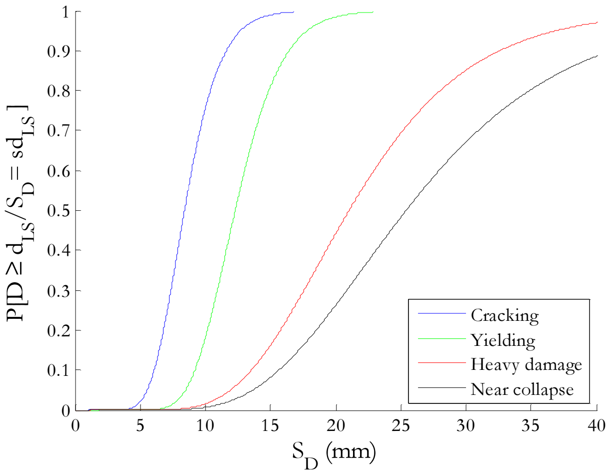


Fig. 6: Fragility Functions for Two Storey Stone Masonry Buildings of Pakistan

The derived fragility functions can be used for the intensity-based (code spectra), scenario-based (historical earthquakes, future expected event) and time-based (loss exceedance curve, annualized losses) earthquake loss assessment on regional scale for developing risk maps, insurance modelling, community earthquake preparedness planning and disaster mitigation in the region.

6. Conclusions

The paper presents calibration, using experimental and numerical investigations, of a nonlinear static analytical displacement-based method, which uses the basic principles of the mechanics of material and structures, for seismic risk assessment of stone masonry buildings of Pakistan. The numerical investigation considered 125 cases study 2D prototype structural models which are analyzed using dynamic analyses with 10 real accelerograms in order to derive mechanical models for the considered building typology taking into account the geometric and material uncertainties as well the record-to-record variability in the capacity evaluation. Controlled Monte Carlo simulation is used to generate random populations of regional building stock and to develop analytical displacement-based fragility functions which can be used to derive damage probability matrices and compute socio-economic losses in the region for risk map development, public awareness and community earthquake preparedness planning in order to mitigate the future expected regional risk.

Acknowledgement

The authors would like to acknowledge the support provided by the European Commission under FP7 through the financing of the SYNER-G research programme, under the framework of which this work has been funded.

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