Development of Displacement-Based Method for Seismic Risk Assessment of RC Building Stock of Pakistan

N. Ahmad[1*], Y.I. Badrashi[2], Q. Ali[3], H. Crowley[4], R. Pinho[5]

[1*] *ROSE School-IUSS Pavia, Via Ferrata 01, 27100 Pavia, Italy* (anaveed@roseschool.it)
[2] *University of Engineering and Technology, Peshawar, Pakistan* (yasir_badrashi@yahoo.com)
[3] *EEC, University of Engineering and Technology, Peshawar, Pakistan* (drqaisarali@nwfpuet.edu.pk)
[4] *EUCENTRE, Pavia, Italy* (helen.crowley@eucentre.it)
[5] *Department of Structural Mechanics, University of Pavia, Pavia, Italy* (rui.pinho@unipv.it)

Abstract

Calibration of an analytical nonlinear static displacement-based method, presented herein, is performed, using experimental and numerical investigation, for the seismic vulnerability and risk assessment of existing reinforced concrete (RC) building stock of Pakistan on regional scale. The method makes use of equivalent single degree of freedom systems, mechanical models, to assess the seismic vulnerability of structures with due consideration of expected uncertainties in geometrical and mechanical properties of the structures besides the uncertainties in seismic demand. A mechanical model is defined completely by knowing the secant vibration period, limit states displacement capacity and viscous damping of the structural material and uses the overdamped displacement spectrum to assess the seismic demand on the structural system. Nonlinear dynamic time history analysis of existing masonry infill RC buildings are performed using natural accelerograms and regularized fiber-based frame elements idealization with site-specific material properties in order to derive the equivalent mechanical models of the considered RC buildings. Controlled Monte Carlo simulation is performed to simulate the regional building stock of RC buildings respecting the regional uncertainties in geometric and material properties which are used then to derive analytical fragility functions and socio-economic loss functions for the considered buildings. These fragility functions can be used to derive damage probability matrices, develop damage scenarios and quantify the socio-economic impacts of regional earthquakes for earthquake preparedness and planning, emergency response and planning, earthquake disaster mitigation and earthquake insurance modeling using intensity-based (code spectra), scenario-based (historical earthquakes), and time-based (annualized losses) loss estimation.

Keywords: Displacement-Based; Nonlinear Static; Mechanical Models; Fragility Functions; RC buildings; Masonry Infill; Pakistan

1. Introduction

In the recent past, with the need for highrise buildings and development of skilled manpower, reinforced concrete structures are fast evolving into the leading construction type in the developing urban areas of Pakistan. However, till recent past, this structural system had been designed for gravity loads only without any consideration for seismic loading or with unknown seismic capacity level due to the lack of expertise of the local engineering and designing community.

Many cases for heavy damages and collapses of RC building stock has been observed in the recent 2005 Kashmir earthquake, in the order of 50-60% of the total RC building stock [1]. Many urban areas which are not affected in the near past by recent earthquakes has significant amount of gravity designed RC buildings. Also, due to unawareness of the society poor quality of construction is still in practice, see Fig. 1 for the reinforcement detailing and column-to-foundation & column-to-floor connections. Such connectivity provisions, among others, lead to the damage and collapse of many RC building stock of northern Pakistan in 2005 Kashmir earthquake [1], [2] & [3]. Thus it is necessary to develop tools for the seismic performance assessment of existing and new-designed RC building stock of the country for earthquake preparedness and planning, insurance modeling, code calibration and decision making in the region in order to mitigate the regional seismic risk.

Fig. 1: Column reinforcement and detailing, column-to-foundation connection (left), spacing of transverse reinforcement (middle), lap-splice provisioned for floor-to-column connectivity (right), observed in the valley of Abbottabad.

2. 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 RC building stock on regional scale. The displacement-based method is originally proposed by [4] and developed for other building typologies of Pakistan [5], [6], [7], [8], [9], and consequently further developed for RC 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. [10] among others. The calibration of the methodology is performed herein in light of the observed damage states of RC building stock in the recent 2005 Kashmir earthquake, see Fig. 2.

Fig. 2: Observed damage states of RC buildings during 2005 Kashmir earthquake: shear cracks in masonry infill, moderately damaged building (left) [11], heavily damaged and collapsed infill with significantly damaged RC columns, extensively damaged building (middle) and collapsed building (right) [3].

2.1 Nonlinear Static SDOF Systems, Mechanical Models, for RC Buildings

The methodology makes use of SDOF system, called a mechanical model, has nonlinear lateral force-displacement response to assess the seismic performance of RC structural systems. The mechanical model simulates the response of the structural system in terms of its displacement capacity, energy dissipation and secant vibration period at different performance levels i.e. damage states, see Fig. 3. In this figure, H_T represents the total building height; h_i represents the i^{th} floor level of buildings, Δ_i represents the lateral displacement and m_i represents the i^{th} floor mass for a given deformed shape of RC 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 structural system at the center of seismic force for a specified deformed shape; K_i represents the initial pre-yield stiffness; F_y represents yielding force; K_{sec} represents secant stiffness, α represents post-yield stiffness.

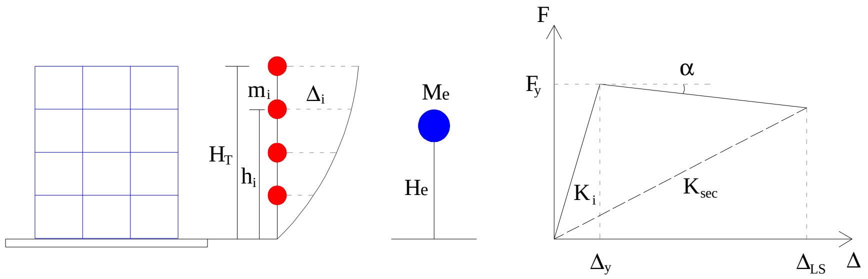

Fig. 3: Nonlinear static single degree of freedom idealization of an RC building.

For seismic assessment of a building, the mechanical model is completely defined by secant vibration period, limit state displacement capacity and viscous damping:

$$T_{LS} = T_y \sqrt{\frac{\mu}{1+\alpha\mu-\alpha}} \qquad (1)$$

$$\Delta_{LS} = \theta_y k_1 H_T + (\theta_{LS} - \theta_y) k_2 h_S \qquad (2)$$

$$\xi_{eq} = 0.05 + 0.565\left(\frac{\mu-1}{\mu\pi}\right) \qquad (3)$$

where T_{LS} represents the limit state secant vibration period; T_y represents the yield period; $\mu = \Delta_{LS}/\Delta_y$ represents the limit state ductility; Δ_y represents the yield limit state displacement capacity; Δ_{LS} represents the post-yield limit states displacement capacity; θ_y represents the yield limit state drift; θ_{LS} represents the post-yield limit states drift; H_T represents height of the building; k_1 and k_2 represent the coefficients to convert the multi degree of freedom (MDOF) structural system to equivalent SDOF system and simulate the displacement capacity at the center of seismic force, which are used also to take into account the record-to-record variability in the displacement capacity evaluation; h_S represents interstorey height of the building; ξ_{eq} represents the equivalent viscous damping of the RC frame buildings recommended by [12]. Eq. (2) considers soft storey mechanism of building at the post-yield limit states which is evident for the considered building typology [1].

2.2 Development of Damage Scenarios 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 (Pf_j). The number of buildings in a given damage state i.e. damage probability matrices (DPM), is obtained, considering the prevailing/observed damage mechanism of existing RC building stock of Pakistan , as follows: undamaged (D0)=1-Pf_1; minor damaged, hairline cracks in masonry infill (D1)=Pf_1-Pf_2; moderate damaged, major cracks in masonry infill with slight cracking of column cover concrete (D2)=Pf_3-Pf_2; major damaged, collapse of masonry infill with heavy damaged concrete column ends (D3)=Pf_4-Pf_3; soft storey collapse of building (D4)=Pf_4-Pf_3, which are used to develop damage scenario, estimate the socio-economic losses of earthquakes and develop regional risk maps for earthquake preparedness planning and risk mitigation in the region.

3. Derivation of Mechanical Models for RC Building Stock of Pakistan

3.1 Characteristics of the Cases Study Buildings

RC building stock are being in practice for around 25 years in the urban areas of Pakistan having mostly two to five stories with the ground floor being utilized for commercial purposes and upper stories have multiple housing units. Major cities like Faisalabad, Islamabad (Capital of Pakistan), Karachi, Lahore, and others practiced RC buildings in the order of 10 to 20% of the total urban building stock [13]. Commercial buildings like plazas, hotels, hospitals etc are constructed mainly of RC frame buildings. The typical building system consists of mostly regular, both in plan and elevation, gravity designed RC moment resisting frame with unreinforced masonry, in cement mortar or lime mortar, infill walls with the most prevailing building dimensions: width ranges from 10m to 20m, length ranges from 10m to 40m and interstorey height of 3m. The buildings are provided with RC slab, comprised of reinforcing steel of 9.5mm or 12.7mm diameter in both directions, having thickness of 120 to 150mm. The buildings are provided with in-plane RC frames 5m apart having beam clear spacings of 4.5 to 5.0m. The typical dimensions adopted for beams are 305mmx350mm to 305mmx460mm, typically reinforced with five to six 19mm to 25.5mm longitudinal bars tied with 9.5mm to 12.7mm stirrups provided at 102mm center-to-center distance, and for columns the typical dimensions are 230mmx305mm to 230mmx610mm, typically reinforced with eight 19mm to 25.5mm longitudinal bars tied with 9.5mm stirrups provided at 150mm to 300mm center-to-center distance. The building rests on shallow foundation with RC isolated footings, with typical dimension of $1.5m^2$ and thickness of 150mm overlain a layer of lean concrete provided over the compacted earth. The compressive strength of masonry infill varies from 2MPa to 5MPa while the concrete, used in beam/column/slab/foundation, compressive strength varies from 10 MPa to 18MPa. The tensile strength of reinforcing bars varies from 230MPa to 300MPa. Building material with relatively high mechanical characteristics can also be observed in the region however the present study adopted lower estimate of the characteristics in order to be conservative in structural system capacity evaluation. The seismic response mechanism of the considered building typology is discussed in the earlier sections herein.

3.2 Structural Modeling of Prototype Cases Study Buildings

A seismic provisioned building code is drafted for new construction in the region since 2007 [14] but which is, nevertheless, not practiced widely, see Fig. 1. The material and structural characteristics of existing buildings provided in the previous section can apply to around 80% of RC building stock of Pakistan and which are considered for the present cases study structural models.

The present study analyzed 200 cases study 2D structural models, using nonlinear time history analysis (NLTHA), from two to five storeys, 50 structural models for each storey, designed with the regional site-specific characteristics properties, as discussed above, considering lognormal random distribution of structural properties. The study aim to derive mechanical models for the considered building typology which will be used then to develop analytical fragility functions for these building systems in the region.

The structural models are prepared in the fiber-based Finite Element Analysis software OpenSees [15]. The masonry infill is modeled using a macro-modeling techniques with in-plane diagonal struts. Many experimental and numerical investigations have been performed on masonry infill RC frames to develop either a finite element approach or an equivalent diagonal strut approach for the development of analytical assessment tools. A very comprehensive literature survey on masonry infill RC frames, experimental and numerical investigations, can be found elsewhere [16]. However, the present study incorporated a new structural modeling technique for masonry infill RC frames which consider the fundamentals of already developed macro-modeling of masonry infill, respecting the regional structural prevailing mechanism and confirming to significant simplicity in modeling in order to reduce the computational cost in the analysis of large regional building stock, see Fig. 4 for the idealization and modeling of masonry infill RC frame. The present masonry infill RC modeling is the further extension and development of the proposed hypothesis of [17] for the considered Pakistani RC building stock and which can be easily extended to the coupled in-plane and out-of-plane dynamic analysis of masonry infill frames [17].

In this modeling hypothesis the RC beams and columns are modeled using the regularized forced-based fiber elements, beamwithhinges force-based fiber element proposed and developed by [18], which is computationally efficient and which avoid the localization and nonobjectivity issues common with softening behavior of RC elements [18]. The elastic part of the beamwithhinges is provided with the crack section stiffness properties.

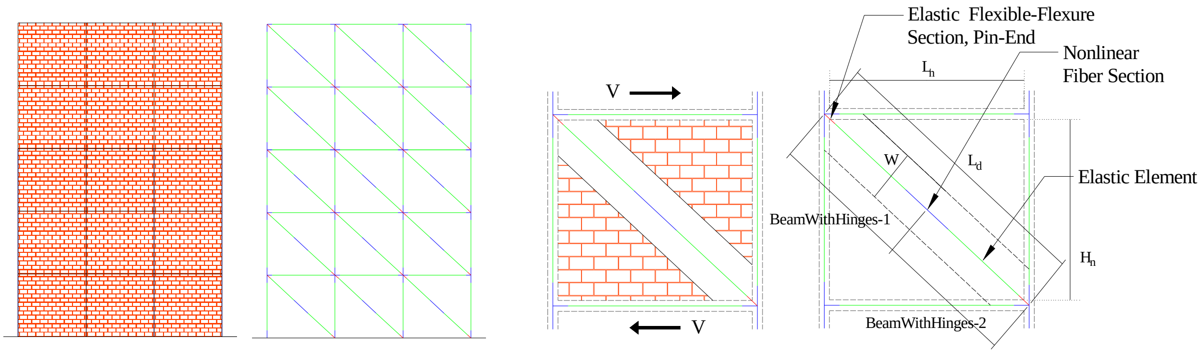

Fig. 4: Structural idealization of masonry infill RC frame wall (left) and details of masonry infill macro-modeling (right).

The masonry infill is modeled using both compression-tension diagonal strut. The provision of only tension-compression diagonal strut, unlike the compression double diagonal struts, is reasonable for the rigid floor analysis, as considered herein due to the observed damage mechanism see Fig. 2, and an ideal option when considering large building stock. Such modeling hypothesis neglect the local damages that occur at the extreme ends of the columns at the contacts with infill. However, the consequences are not dramatically affecting the stiffness and strength of the global system which is mainly contributed by the masonry infills. Nevertheless, calibration with the experimental work on the considered building system is required to quantify, if significant, the effects of local phenomenon and develop further the numerical modeling hypothesis set forth herein.
The strut is modeled using the proposal of [17] with two beamwithhinges connected at the middle with the elastic, significantly flexible, section at the extreme ends, to simulate pin-end condition, and nonlinear fiber-section in the middle, which simulate the nonlinear behavior of masonry infill. A nonlinear axial stress-strain hysteretic rule (for the middle nonlinear fiber section hinge), cross sectional area of the strut and masonry moduli (used for the elastic part of strut) are required to completely define the masonry infill model for nonlinear static and dynamic seismic analysis. A number of analytical models are available to compute the required width of the strut for different types of masonry typology [20],[20],[22], the present study used the model developed by [23]:

$$W = \frac{0.175 L_n}{\cos\theta (\lambda H)} \tag{4}$$

$$\lambda = \left(\frac{E_m b \sin 2\theta}{4 E_c I_c H_n} \right) \tag{5}$$

where W represents the width of diagonal strut; L_n represents the net horizontal dimension of infill panel; $\theta = \arctan(H_n/L_n)$; H_n represents the infill height; H represents the column height; b represents the thickness of the infill; I_c represents the moment of inertia of the exterior frame columns; E_c represents the concrete Young modulus; E_m is the masonry Young modulus in the diagonal direction [24]. Another proposal to compute the Young modulus at any inclination of masonry elements is given by [25]:

$$E_{m,\theta} = \left[\frac{\cos^4\theta}{E_{mh}} + \frac{\sin^4\theta}{E_{mv}} + \cos^2\theta . \sin^2\theta \left(\frac{1}{G} - 2\frac{\nu}{E_{mv}} \right) \right]^{-1} \tag{6}$$

where $E_{m\theta}$ represents the Young modulus of the equivalent diagonal strut; E_{mh} represents the masonry Young modulus in the horizontal direction; E_{mv} represents the masonry Young modulus in the vertical direction; G represents the masonry shear modulus; ν represents the poisson ratio.
A tri-axial stress-strain hysteretic rule, also developed by [26] , is assigned to the nonlinear hinge, see Fig. 4, for which the maximum axial strength is obtained using the empirical strength model proposed by [27], with the cracking strength 60% of the maximum axial strength:

$$P_\theta = \frac{a.L_n b f_c}{\cos\theta} \tag{7}$$

where P_θ represents the axial strength of diagonal strut; L_n represent the horizontal length of infill; b represents the infill thickness; f_c represents the diagonal compression strength; a represents the reduction coefficient varies between 1 & 1.3. In the absence of the diagonal strength of panel, [17] recommend to consider a as 1 and f_c considered as the shear strength of masonry which is a widely available parameter for masonry and gives the strength roughly close to the value obtained using the model [28] for the diagonal shear failure of masonry infill.
Eight concrete cylinders, prepared in mix design to respect the existing field practice, are tested in the structural laboratory of UET Peshawar to obtain the basic material properties of concrete, mainly the compressive strength (mean of 17MPa with β of 0.24MPa, where β represents the logarithmic standard deviation), strain at maximum strength (mean of 0.0026 with β of 1.05) and strain at crushing (mean of 0.0087 with β of 1.00). Which are required for the definition of stress-strain constitutive law of concrete in the fiber-based section modeling of RC structural members.
The experimental work carried out by [29] on four masonry panel is used to compute the diagonal strength of brick masonry in cement-khaka mortar used in Eq. (7). Additionally, tests are performed recently on three masonry panels, in cement mortar, at the UET Peshawar. A mean value of diagonal strength, considering all seven panels, is obtained to be 4.56% of the compressive strength of masonry with β of 0.2336% of the compressive strength of masonry. Fig. 5 shows the damage pattern of the tested two classes of masonry panels: in cement-khaka mortar (left) and cement mortar (right), which is comparable with the observed behavior of infill during the 2005 Kashmir earthquake, see Fig. 2 among others.

Fig. 5: Evaluation of diagonal strength of masonry panel, from [29] (left) and test conducted at the UET Peshawar (right).

Once all the 200 structural models are prepared and designed with the site specific structural characteristic properties, randomly generated using Monte Carlo simulations, a nonlinear static conventional pushover analysis of all the models is performed in order to check the numerical stability and soundness of the modeling technique. For simplicity reason and due to the observed response of the considered building typology, uniform load pattern is considered. Which is also important to compute the possible maximum strength of the structural models that can provide guidance on the appropriate scaling of the accelerogram employed for the NLTHA, see Fig. 6 for capacity curves of three storey buildings.

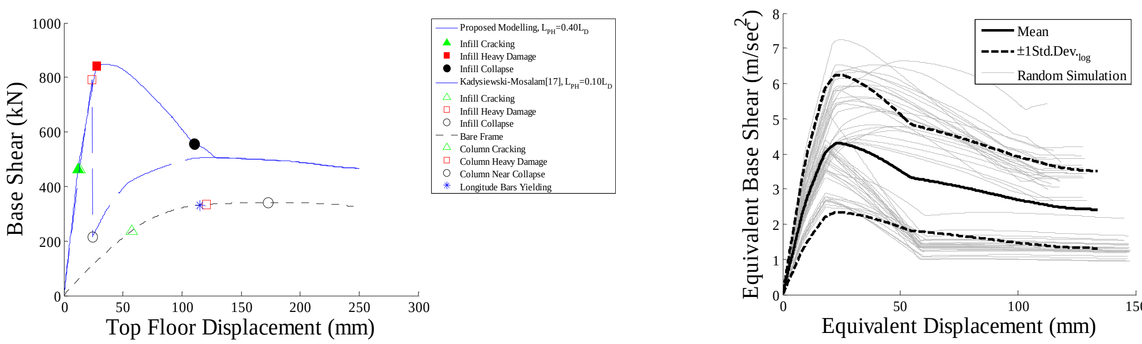

Fig. 6: Capacity curves of the considered masonry infill RC buildings, the effect of hinge length on the infill capacity and comparison with bare frame (left), capacity curves for randomly generated three storey masonry infill RC frames (right).

The capacity curves also show the structural model response even after the infill failure, which corresponds to the near collapse limit state of the considered building typology, in order to check the numerical stability of the models only. The damage states of infill frame is dictated by the damage

states of the masonry infill at the ground floor. For bare frame, the concrete strain limits, obtained experimentally, and longitudinal steel yield strain are used to described the damage state of the ground floor columns. For bare frame, the shear failure of ground floor columns, although didn't investigated herein, may reach earlier than the higher limit states, near collapse state as shown in Fig. 6, of concrete are achieved. Thus, both infill and bare frames may have roughly the same displacement capacity at the near collapse limit states. Thus it can be concluded that for the considered gravity designed RC frames the inclusion of masonry infill increases the structural strength and hence the seismic capacity of the system.
It is observed that the diagonal strut hinge length has a significant effect on the nonlinear response of the structural models. A smaller hinge length, 10% of the diagonal strut length as proposed by [17], can result in an abrupt drop in the strength of the system which is although possible for week masonry infill, like perforated masonry units, is extremely under estimating the response of a ductile masonry infill which contribute significantly even if the maximum strength of the infill is achieved. Thus prudently somewhat larger hinge length is adopted, 40% of the diagonal strut length, in order to obtain some reasonable ductility level, in the range of 3 to 5 which is common for masonry with RC columns confinement [30], for the considered structural system. Alternatively, a horizontal sliding shear spring is added along with the infill panel, considering the brittle failure of the axial strut, which contribute to the lateral load carrying capacity once the strut attains its maximum axial strength, [20], [21], [22], [31]. This hypothesis although validated through many cases studies however does not take into account the masonry strength degradation effects after the peak strength of the infill is reached. Nevertheless, experimental work is required on the considered masonry infill RC frame system in order to investigate whether the provision of additional horizontal spring is necessary for the proposed modeling or if alternatively a reasonable length of nonlinear hinge can be selected for the infill to avoid the provision of additional sliding spring and make the overall modeling simple.

3.3 Nonlinear Time History Analysis of Cases Study Structural Models

The prototype cases study structural models are analyzed dynamically through NLTHA with 10 natural accelerograms extracted from the PEER NGA data base with the mean spectrum compatible to EC8 Type I C-soil spectrum [32], previously used [7]. The accelerograms are linearly scaled in order to observe the post-yield response of all the structural models which is aimed to derive static SDOF systems, mechanical models, for the considered building typology and retrieve the dynamic characteristics of the considered structural models at different damage states.
The scope of the dynamic analysis is to compute the equivalent base shear and equivalent displacement demand at the different limit states of masonry infill on the ground floor using the proposed SDOF derivation of [12] and compute the dynamic characteristic of the buildings, mainly the secant vibration period and the limit state displacement capacity with due consideration of record-to-record variability for the development of Eq. (1) & (2):

$$T_y = 2\pi\sqrt{\frac{\Delta_{eq}}{VB_{eq}}} \tag{8}$$

where Δ_{eq} represents the equivalent displacement and VB_{eq} represents the equivalent base shear obtained from the normalization of the floor displacements and base shear over the deformed shape and seismic mass participation of the building [12]. The vibration periods obtained for the considered cases study buildings are used to develop an empirical period model for future applications:

$$T_y = a.\exp(\pm\varepsilon\beta)H^b \tag{9}$$

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, which shows relatively well correlation than b set to 1.0, obtained for 50 three storey structural models with record-to-record variability is shown in Fig. 7 (left). The empirical model for period computation, Eq. (9), developed for the considered building stock is also shown in Fig. 7.
Additionally, the derived SDOF system properties are used to develop the equivalent displacement capacity model, Eq. (2). 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 analytical model proposed by [33]:

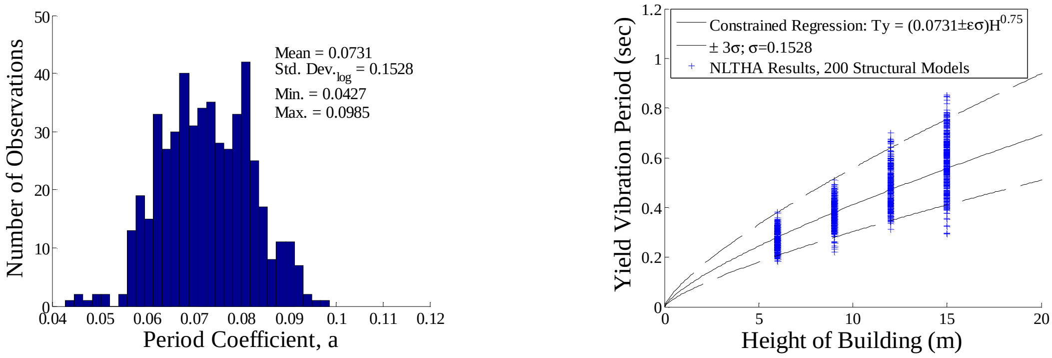

Fig. 7: Likelihood of period coefficient for three storey RC buildings (left), yield vibration period for all cases study buildings and the development of an empirical model for period computation of considered RC building stock (right).

$$\theta_{LS} = \frac{L}{H} - \sqrt{(1-\varepsilon_w)^2\left(1+\left(\frac{L}{H}\right)^2\right)-1} \tag{10}$$

where θ_{LS} represents the limit state interstorey drift corresponding to different damage states of masonry infill; L represents the bay width; H represents the interstorey height; ε_w represents the limits states axial strain of infill strut. This gives the interstorey drift of 0.2267%, 0.4535% and 2.2708% for axial strut strain of 0.001 mm/mm, 0.002 mm/mm and 0.01 mm/mm which corresponds to cracking, heavy damage and collapse limit states of masonry infill. For k_1, mean value of 0.7886 with β 0.0466, min 0.6972 and max 0.8199 for two storey; mean value of 0.7318 with β 0.0438, min 0.6530 and max 0.7646 for three storey; mean value of 0.7001 with β 0.0531, min 0.6293 and max 0.7534 for four storey; mean value of 0.6563 with β 0.0639 min 0.5693 and max 0.6988 for five storey are obtained. k_2 is theoretically 1.00 but is also considered to take in to account the record-to-record variability in the displacement capacity at post-yield limit states.

4. Derivation of Displacement-Based Analytical Fragility Functions

The state-of-the-art, conceptual and explicit procedure, in contrary to the existing conventional procedures e.g. [10] among others, proposed by [34] for the derivation of analytical fragility functions for masonry buildings is used herein to derive displacement-based fragility functions for RC buildings of Pakistan. In the first step, controlled Monte Carlo simulation is used to generate random SDOF systems 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 the 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 for building damage states and economic loss functions, using the loss model of [35], which can be used for the regional seismic risk modeling, assessment and mitigation activities.

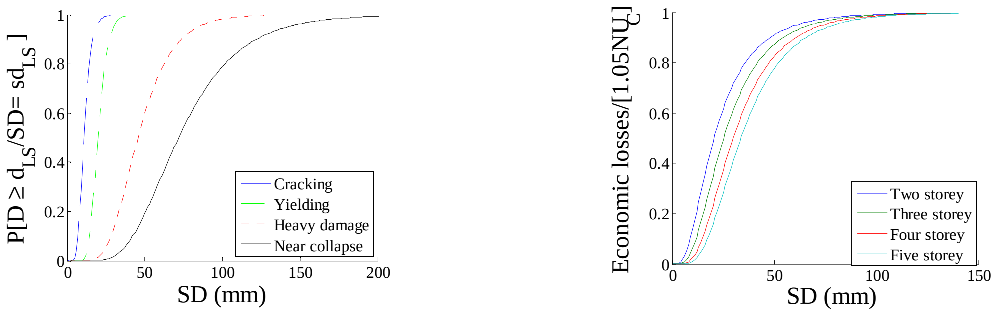

Fig. 8: Fragility functions for three storey buildings (Left) and economic loss functions for considered building stock (Right).

5. Conclusion and Future Development

The paper carried out the calibration of a nonlinear static analytical displacement-based method for seismic risk assessment and loss estimation of RC building stock of Pakistan on regional scale. Numerical investigation of 200 cases study 2D prototype structural models is performed using dynamic analyses with 10 natural 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 develop damage scenarios and compute the socio-economic losses in the region for risk map development, public awareness and community earthquake preparedness planning, insurance modeling and code calibration in order to mitigate the future expected regional risk. The masonry infill RC frame modeling technique and the static mechanical models derived herein can be improved further once more detailed and reliable test data on materials and structures are made available. Also, field survey is required to analyze in detail the current construction practices in the region in order to take into account the construction improvement in the derivation of mechanical models and fragility functions. Extension to high rise RC buildings is considered for future research.

References

[1] Naseer, A., Naeem, A., Hussain, Z., and Ali, Q., *Observed Seismic Behavior of Buildings in Northern Pakistan During the 2005 Kashmir Earthquake*, Earthquake Spectra, Vol.26, No. 2, pp. 425-449. 2010.

[2] Rossetto, T., and Peiris, N., *Observations of damage due to the Kashmir earthquake of October 8, 2005 and study of current seismic provisions for buildings in Pakistan,* Bulletin of Earthquake Engineering Vol.7, No.3, pp.681-699, 2009.

[3] Peiris, N., Rossetto, T., Burton, P., and Mahmood, S., *EEFIT Mission: October 8, 2005 Kashmir Earthquake,* Technical Report, Earthquake Engineering Field Investigation Team, Institution of Structural Engineers, London, United Kingdom, 2008.

[4] Crowley, H., Pinho, R., and Bommer, J.J., *A Probabilistic Displacement-Based Vulnerability Assessment Procedure for Earthquake Loss Estimation*, Bulletin of Earthquake Engineering Vol.2, No.8, pp. 173-219, 2004.

[5] Ahmad, N., Crowley, H., Pinho, R., and Ali, Q., *Simplified Formulae for the Displacement Capacity, Energy Dissipation and Characteristic Vibration Period of Brick Masonry Buildings,* Proc. of the International Masonry Society, Vol.11, No.2, pp. 1385-1394, 2010.

[6] Ahmad, N., Crowley, H., Pinho, R., and Ali, Q., *Displacement-Based Earthquake Loss Assessment of Masonry Buildings in Mansehra City, Pakistan,* Journal of Earthquake Engineering Vol.14, No.S1, pp.1-37, 2010.

[7] Ahmad, N., Crowley, H., Pinho, R., and Ali, Q., *Displacement-Based Seismic Risk Assessment of Stone Masonry Buildings of Pakistan*, Proc. of the Third Asian Conference on Earthquake Engineering, Bangkok, Thailand, Paper No. ACEE-P-101, 2010.

[8] Ahmad, N., Crowley, H., Pinho, R., and Ali, Q., *Displacement-Based Earthquake Loss Assessment of Adobe Buildings of Pakistan*, Proc. of the Sixth International Structural Engineering and Construction Society, Zurich, Switzerland, Paper No. S2_S54, 2011.

[9] Ahmad, N., Ali, Q., Crowley, H., and Pinho, R., *Displacement-Based Seismic Performance Evaluation of Dhajji Structural Systems*, Proc. of the Eleventh North American Masonry Conference, Minnesota, USA, Paper No. 89, 2011.

[10] HAZUS-99., *Earthquake Loss Estimation Methodology*, Technical Manual, FEMA, Washington D.C., USA, 1999.

[11] Javed, M., Naeem, A., Penna, A., and Magenes, G., *Behavior of Masonry Structures during the Kashmir 2005 Earthquake*, Proc. Of the First European Conference on Earthquake Engineering and Seismology, Geneva, Switzerland, Paper no.1093 (CD-ROM), 2006.

[12] Priestley, M.J.N., Calvi, G.M., Kowalsky, M.J., *Displacement-Based Seismic Design of Structures*, IUSS Press, Pavia, Italy, 2007.

[13] Badrashi, Y.I., Ali, Q., and Ashraf, M., *Reinforced Concrete Buildings in Pakistan*, Technical Report, World Housing Encyclopedia, Earthquake Engineering Research Institute, Oakland, CA, USA, Report No. 159, 2010.

[14] BCP., *Building Code of Pakistan-Seismic Provisions-2007,* Ministry of Housing and Works, Islamabad, Pakistan, 2007.
[15] McKenna, F., Fenves, G.L. and Scott, M.H., *Open system for earthquake engineering simulation (OpenSees): version2.2.2*', *URL* http://opensees.berkeley.edu, 2010.
[16] Griffith, M., *Seismic Retrofit of RC Frame Buildings with Masonry Infill Walls: Literature Review and Preliminary Case Study,* Scientific and Technical Report, European Commission Joint Research Centre, Ispra, Italy, 2008.
[17] Kadysiewski, S., and Mosalam, K.M., *Modeling of Unreinforced Masonry Infill Walls Considering In-Plane and Out-of-Plane Interaction*, Pacific Earthquake Engineering Research Center, PEER, Report No. 2008/102, 2009.
[18] Scott, M.H. and Fenves, G.L., *Plastic Hinge Integration Methods for Force-Based Beam-Column Elements*, Journal of Structural Engineering, ASCE, Vol.132, No. 2, pp.244-252, 2006.
[19] Calabrese, A., Almeida, J.P., and Pinho, R., *Numerical Issues in Distributed Inelasticity Modeling of RC Frame Elements for Seismic Analysis,* Journal of Earthquake Engineering Vol.14, No.S1, pp.38-68, 2010.
[20] Smyrou, E., Blandon-Uribes, C., Antoniou, S., Pinho, R., and Crowley, H., *Implementation and Verification of a Masonry Panel Model for Nonlinear Pseudo-Dynamic Analysis of Infilled RC Frames,* Proc. Of the First European Conference on Earthquake Engineering and Seismology, Geneva, Switzerland, Paper no. 355 (CD-ROM), 2006.
[21] Smyrou, E., *Implementation and Verification of a Masonry Panel Model for Nonlinear Dynamic Analysis of Infilled RC Frames,*Master Thesis, ROSE School – IUSS Pavia, Pavia, Italy.
[22] Crisafulli, F.J., *Seismic Behavior of Reinforced Concrete Structures with Masonry Infills,* PhD Thesis, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.
[23] Mainstone, R.J., *On the Stiffness and Strength of Infilled Frames,* Proc. Of the Institute of Civil Engineers, London, United Kingdom, 1971.
[24] Kappos, A.J., Stylianidis, K.C., and Michailidis, C.N., *Analytical Models for Brick Masonry Infilled R/C Frames Under Lateral Loading,* Journal of Earthquake Engineering Vol.2, No.1, pp. 59-87, 1998.
[25] Sacchi, M.G., and Riccioni, R., *Comportamento Statico e Sismico Delle Strutture Murarie* (in Italian), CLUP Editore, Milano, Italy, 1982.
[26] Dolsek, M., and Fajfar, P., *Mathematical Modelling of an Infillled RC Frame Structure Based on the Results of Pseudo-Dynamic Tests,* Earthquake Engineering and Structural Dynamics, Vol.31, No. 6, pp.1215 –1230, 2002.
[27] Fardis, M.N., Calvi, G.M., *Effects of Infill on the Global Response of Reinforced Concrete Frames,* Proc. Of the Tenth European conference on Earthquake Engineering, Vienna, Austria, Vol.4, pp.2893-2898, 1994.
[28] Bertoldi, S.H., Decanini, L.D. and Gavarini, C., *Telai Tamponati Soggetti Ad Azione Sismica, un Modello Semplificato: Confronto Sperimentale e Numeric* (in Italian), Atti del 6° Convegno Nazionale, Perugia, ANIDIS, Vol.2, pp.815-824, 1993.
[29] Javed, M., *Seismic Risk Assessment of Unreinforced Brick Masonry Building System of Northern Pakistan*, PhD-Thesis, Civil Engineering Dept., NWFP University of Engineering and Technology Peshawar, Pakistan, 2008.
[30] Tomazevic, M., Earthquake-Resistant Design of Masonry Buildings-Innovation in Structures and Construction Vol. 1., Imperial College Press, London, UK, 1999.
[31] SeismoSoft, *SeismoStruct – A Finite Element Computer Program for Static and Dynamic Nonlinear Analysis of Frame Structures* , URL: http://www.seismosoft.com, Pavia, Italy, 2006.
[32] Comite Europeen de Normalization (CEN)., *Eurocode 8: Design of Structures for Earthquake Resistance, Part1: General Rules-Seismic Actions and General Requirements for Buildings*, EN 1998-1-3, CEN, Brussels, 2004.
[33] Magenes, G., and Pampanin, S., *Seismic Response of Gravity-Load Design Frames With Masonry Infills,* Proc. of the Thirteenth World Conference on Earthquake Engineering, Vancouver, Canada, Paper No.4004, 2004.
[34] Ahmad, N., Crowley, H., Pinho, R., and Ali, Q., *Derivation of Displacement-Based Fragility Functions for Masonry Buildings,* Proc. of the Fourteenth European Conference on Earthquake Engineering, Ohrid Macedonia, Paper No.327, 2010.
[35] Bal, I.E., Crowley, H., Pinho, R., and Gulay, G., *Detailed Assessment of Structural Characteristics of Turkish RC Building Stock for Loss Assessment Models,* Structural Dynamics and Earthquake Engineering Vol.28, No.10-11, pp.914-932, 2008.