Simplified formulae for the displacement capacity, energy dissipation and characteristic vibration period of brick masonry buildings

AHMAD, NAVEED[1]; CROWLEY, HELEN[2]; PINHO, RUI[3]; ALI, QAISAR[4]

ABSTRACT

The paper presents the calibration of a displacement-based method, presented herein, for seismic vulnerability assessment of masonry building stock on regional scale. The method makes use of equivalent single degree of freedom (SDOF) systems 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. The SDOF system is defined completely by knowing the characteristic vibration period, limit states displacement capacity and the characteristic energy dissipation of the structure. Nonlinear dynamic time history analysis of 3D masonry building classes are performed using equivalent frame method to derive the equivalent SDOF systems of masonry buildings. The SDOF systems take into account the stiffness and strength degradation due to cyclic and in-cyclic response, higher mode effects and torsional response of the buildings. Furthermore, the available experimental data on masonry walls is obtained in order to define the limit state deformation capacity at different performance levels and the characteristic energy dissipation of masonry buildings. Simplified formulae are proposed for the aforementioned characteristic properties of masonry buildings.

Keywords: Unreinforced masonry, vulnerability assessment, displacement-based, equivalent frame method,

1 INTRODUCTION

A simplified nonlinear displacement-based method, developed for reinforced concrete buildings [1], is presented herein. The method uses the basic principle of the mechanics of materials and structures to assess the seismic vulnerability of classes of structures on regional scale taking into account the expected variability in the geometrical and mechanical properties of the structural systems. The emerging concepts of performance based engineering for masonry made it possible to calibrate the aforementioned methodology for such building systems as well. The calibration of the method is performed for residential unreinforced brick masonry (URBM) buildings in the urban areas of Pakistan. Nonlinear dynamic time history analysis of 7 case study buildings is performed in a 3D space, using a simplified method, in order to obtain the characteristic vibration periods of masonry buildings. Experimental data is obtained from the available literature on the nonlinear cyclic response

[1] PhD Candidate, ROSE School, IUSS Pavia, 27100 Pavia, Italy, anaveed@roseschool.it
[2] Researcher, EUCENTRE, 27100 Pavia, Italy, helen.crowley@eucentre.it
[3] Assistant Professor, Department of Structural Mechanics, University of Pavia, Italy, rui.pinho@eucentre.it
[4] Director, Earthquake Engineering Center, NWFP University of Engineering and Technology Peshawar, Pakistan, drqaisarali@nwfpuet.edu.pk

of URBM shear walls and presented. The data is analyzed to understand the deformation capacity of URBM walls at different performance levels and their energy dissipation characteristics for cyclic loads. The aforementioned properties of URBM buildings are formulated in a simplified expression for their use in the seismic vulnerability assessment of URBM buildings on regional scale for the expected seismicity [2].

2 DISPLACEMENT-BASED EARTHQUAKE LOSS ASSESSMENT (DBELA)

The DBELA method uses the basic concept of idealizing the nonlinear behavior of a building as an equivalent SDOF linear system with a bi-linear force displacement response, Figure 1. In this figure, H represents the total height; h_i represents the i^{th} floor height, Δ_i is the lateral displacement and m_i represents the i^{th} floor mass for a given deformed shape of the building; Me and He are 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 is the initial pre-yield stiffness; F_y is the yielding force; K_{sec} is the secant stiffness; and α is the ratio of post- to pre-yield stiffness. For any limit state the system vibrates linearly at secant period and having viscous damping. Viscous damping represents the equivalent damping of a building at the considered limit state i.e. elastic damping for pre-yield case and hysteretic damping besides the elastic one for post-yield cases. This SDOF system represents the actual characteristics of a building in terms of its equivalent displacement and the energy dissipation of that building for a given seismic demand.

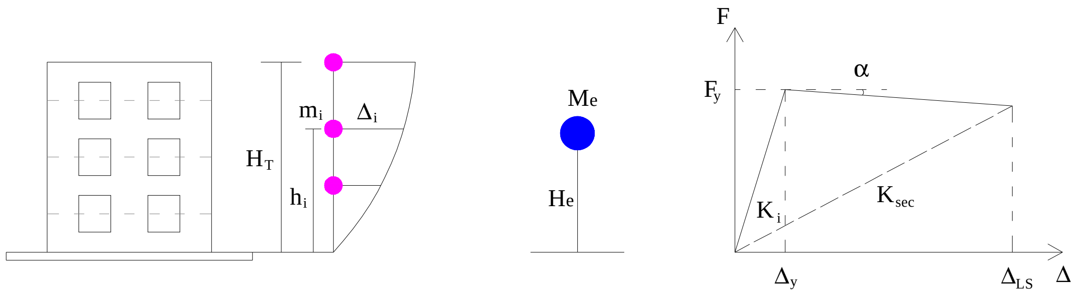

Figure 1. Single degree of freedom idealization of a building

The first step of the method is the generation of a random population of buildings which should represent the whole urban building stock. Monte Carlo simulation is used to generate thousands of buildings, each with different geometrical and material properties; the variability of each property being defined a priori using a complete probabilistic distribution with prescribed mean and coefficient of variation (c.o.v.). Once the population has been generated, the periods of vibration besides the limit states displacement capacity corresponding to different performance levels are estimated for each building within the population using (1):

$$T_{LS} = T_y \cdot \left(\frac{\mu_{LS}}{1 + \alpha \cdot \mu_{LS} - \alpha} \right)^{\frac{1}{2}} \quad (1)$$

where T_y is the yield vibration period at the first limit state; T_{LS} and $\mu_{LS}=\Delta_{LS}/\Delta_y$ are the characteristic vibration period and ductility level at the post yield limit states. The factor α is calibrated from the nonlinear dynamic time history analysis of the buildings herein which considers the reduction of post-yield strength due to torsional response of the building. The stiffness and strength reduction due to cyclic response is considered already through the use of μ_{LS} and T_y, which will be clarified in the following sections. Any suitable criterion can be used for the definition of different performance levels for buildings e.g. no-damage, slight damage, moderate damage, extensive damage, and complete/near collapse states

The next step of the method is to assess the vulnerability of each generated building for a given seismic demand. The seismic demand on buildings is defined using elastic displacement response spectrum using any suitable ground motion prediction equation. The comparison of the structural displacements is carried out at their corresponding vibration periods for different pre-defined limit states considering the system energy dissipation: considered by lowering the elastic displacement response spectrum using an appropriate reduction factor e.g. the reduction factor (2) proposed by [3]:

$$\eta = \left(\frac{7}{2+\xi_{eq}}\right)^{\frac{1}{2}} \qquad (2)$$

where η is the displacement spectral reduction factor and ξ_{eq} is the equivalent viscous damping (provided in percent) of the system at a given limit state. For a response spectrum, the displacement demand at a specified limit state vibration period of each building can be compared with its displacement capacity at that limit state; the sum of all the buildings with displacement capacity lower than the displacement demand divided by the total number of buildings gives an estimation of the probability of exceeding the considered limit state. The sample size is gradually reduced from one limit state to the next by removing the buildings which do not exceed the previous limit state. Once the probability of exceedance at different limit states are obtained, the percentage of buildings in each damage state can be easily identified, which can be used to estimate the expected socio-economic impacts of the considered seismicity in the region [2].

3 A METHOD FOR SIMPLIFIED NONLINEAR STATIC AND DYNAMIC ANALYSIS OF MASONRY BUILDINGS (SD-SAM)

The method used for the dynamic analysis of masonry buildings in the present study is based on the equivalent frame idealization of the buildings with rigid offsets as proposed by [4] for the simplified global analysis of masonry buildings. However, the present study formulates the method with simplified constitutive laws for frame elements representing piers and spandrels which can be applied to any type of masonry buildings i.e. unreinforced, reinforced and confined, with regular openings and rigid or flexible floors. The method uses the idea of modelling masonry spandrels and piers as one dimensional beam-column elements with bending and shear deformation with infinitely stiff joint element offsets at the ends of the pier and spandrel elements, Figure 2.

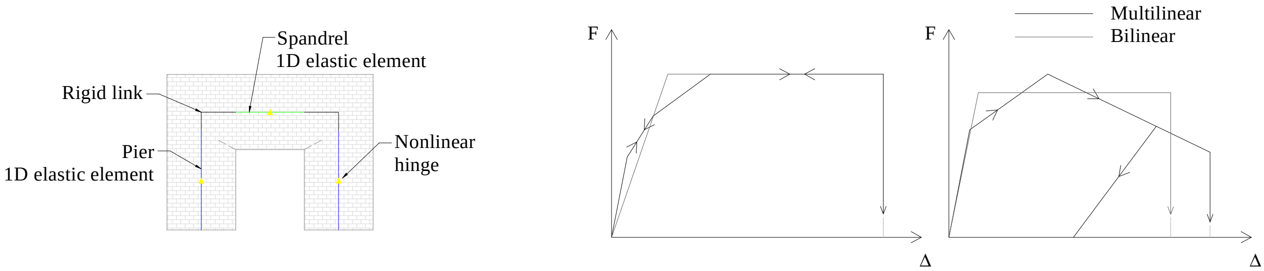

Figure 2. Equivalent frame idealization (left) and nonlinear response of frame element (right)

A simple criterion is used for the identification of the effective deformable length of pier: taking the intersection of the element with a line making a 30 degree angle with the corners of the openings. Each of the elements are provided with hinge that respond only in the in-plane direction against the shear demand in the elements respectively. The hinges are assigned with appropriate inelastic force-displacement response for predicted ultimate mechanism. The frame element responds in flexure bending and fails when its ultimate capacity is reached in shear. The total chord rotation of the element at any instant is the summation of the bending and shear deformation developed in the element. Deformation due to additional in-plane local mechanisms: slippage at bed joints and rigid-body rotations, can also be taken into account.

The possible failure mechanisms considered for unreinforced masonry piers are the flexure or rocking failure, diagonal shear cracking, and shear sliding using the following strength models:

$$V_f = \frac{p \cdot D^2 \cdot t}{2 \cdot H_P \cdot \psi} \cdot \left(1 - \frac{p}{k \cdot f_u}\right) \tag{3}$$

$$V_d = \frac{f_{tu} \cdot D \cdot t}{b} \cdot \left(1 + \frac{p}{f_{tu}}\right)^{\frac{1}{2}} \tag{4}$$

$$V_S = \frac{D \cdot t \cdot (1.5 \cdot c + \mu \cdot p)}{\left(1 + 3 \cdot \frac{c \cdot H_P \cdot \psi}{p \cdot D}\right)} \tag{5}$$

$$k_{c/\mu} = \frac{1}{\left(1 + 2 \cdot \mu \cdot \frac{\Delta_y}{\Delta_x}\right)} \tag{6}$$

where V_f is the ultimate strength for flexure/rocking failure; D and t are the length and thickness of the pier; $p=P/(D.t)$ is the mean vertical stress due to axial load P; H_P is 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 is the compressive strength of the masonry; k is 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} is the diagonal tensile strength, 4% of f_u for Pakistani masonry; 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 is the sliding shear strength; μ and c represent the coefficient of friction and cohesion of masonry as global strength parameters. A correction factor (6) can be used ($\mu=k_{c/\mu} \cdot \mu_{lab}$, and $c=k_{c/\mu} \cdot c_{lab}$) for the laboratory obtained value. The parameters Δ_x and Δ_y are the length and height of the brick unit respectively. A very detailed discussion on the appropriate uses of the above strength models can be found in [5]. The shear strength of the spandrels is computed using the following strength models:

$$V_t = h_{sp} \cdot t \cdot f_{v0} \tag{7}$$

$$V_P = \frac{N_P \cdot h_{sp}}{L_{sp}} \cdot \left(1 - \frac{N_P}{0.85 \cdot f_{hu} \cdot h_{sp} \cdot t}\right) \tag{8}$$

where V_t is the shear strength of spandrel effectively bonded at the ends; h_{sp} is the spandrel section height; t is the spandrel thickness; L_{sp} is the length of spandrel above the opening and f_{v0} is the shear strength in absence of compression on bed joints and can be taken as 2/3 of f_{tu}. The second mechanism, V_P, considers the maximum resisting moment associated to the flexure mechanism in order to resist the tension actions in the spandrel. N_P here is the minimum of the tensile strength of the horizontal element, lintel if any, and the value $0.4f_{hu}h_{sp}t$, where f_{hu} is the compression strength of masonry in the horizontal direction in the plane of the wall. If the building has RC ring beams and/or band beams, they should be considered as rigid except where openings are found on the top and bottom sides of the element.

The nonlinear behaviour of the frame element can be approximated as trilinear or bilinear. In case of bilinear idealization, the strength of elements should be reduced by 10% for the consideration of strength and stiffness degradation due to cyclic response [5]. The element cyclic response should be considered with the hysteretic energy dissipation characteristics. A multilinear or bilinear elastic response can be assumed for flexure/rocking mechanism with limited or conservatively no hysteretic energy dissipation characteristics. The ultimate drift level of 0.4% is recommended for pier responding in shear and 0.8% for flexure rocking mechanism [4]. The possible choices for nonlinear behaviour of spandrels will be elastic-plastic-brittle, elastic-brittle, and elastic-perfectly-plastic [4]. The ultimate drift of 0.3% is recommended for spandrels. The yield drift of nonlinear hinge can be approximately obtained from the shear deformation (in case of shear mechanism), in addition to rigid-body rotation (in case of rocking mechanism), for 50% cracked section shear modulus with effective shear area [4].

For the reliability check of the proposed modelling approach, the method is applied to a two storey masonry building with flexible floor [4], in order to reproduce its capacity curve. The door wall of the building is modelled using OpenSees [6]. Both pier and spandrel elements are assigned with trilinear and bilinear nonlinear force displacement response, defined using the above recommendations. The yield drift is obtained using the 50% cracked section properties with lateral load capacity at the cracking limit state, for trilinear idealization, and at the yielding limit state, for bilinear idealization, of elements. The predicted response using the proposed formulation is compared with the experimental results which is found satisfactory in terms of equivalent yield displacement, maximum lateral strength, and ultimate damage state of the building, see Figure 3.

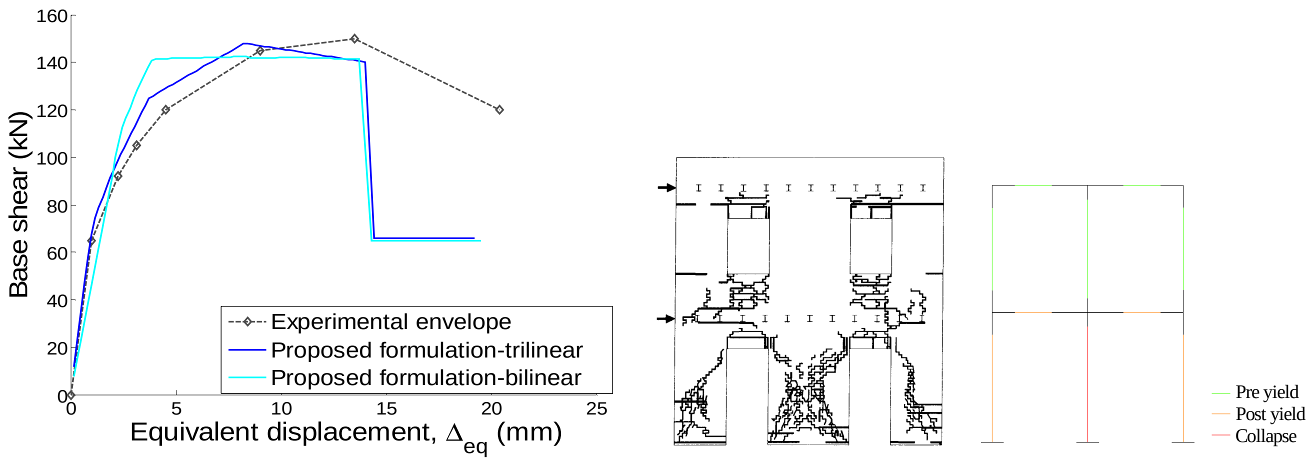

Figure 3. Comparison of the experimental and predicted response of the wall, left: capacity curve and right: ultimate damage state of the wall.

4 CHARACTERISTIC PROPERTIES OF BRICK MASONRY BUILDINGS

4.1. Vibration periods of the buildings

The nonlinear dynamic analysis of 7 case study 3D buildings, one to three storeys representing typical Pakistani urban masonry buildings, are carried out using 60 real acceleration time histories randomly extracted from the PEER NGA database. The recordings were selected with a moment magnitude of 6 to 7.5 within the rupture distance of 10 to 100 km having PGA values ranging from 0.07g to 0.82g. The analysis were carried out for the case study buildings with linearly scaled accelerograms in order to obtain complete capacity curves for buildings taking into account the record-to-record variability in response evaluation. The URBM buildings in the urban areas of Pakistan have a lintel beam below the spandrels and an RC slab above the spandrel with or without RC ring beams. Thus, it was decided to use stiffer spandrel elements for the analyzed buildings. Elastic-perfectly-plastic nonlinear behaviour of frame element in shear is considered using uni-axial hysteretic material, available in OpenSees [6], response with unloading and re-loading stiffness degradation as a function of ductility level in the element. The displacement bounds of a frame element on both negative and positive sides are defined, using MinMax material [6], in order to discard the element contribution beyond the bounds which corresponds to the failure of the element. The hysteretic response of the frame element is considered such that the energy dissipation of the element in each cycle is equal to the experimentally observed value for masonry walls, with beta=0.6 [6]. The energy dissipation characteristics of masonry walls are discussed in the following section.

The scope of the dynamic analysis is to compute the equivalent capacity curves of the buildings and from these curves identify the lateral force and displacement at the first limit state i.e. minor cracks in the masonry elements and compute the vibration period of the buildings at that point taking into account the actual deformed shape and the mass participation of the buildings. A simplified method is used to compute the pre-yield and at yield vibration periods of all the buildings from their capacity curves. Once the data were obtained for all buildings for the base shear force and lateral floor displacement at the peak displacement response, the data were converted to the equivalent properties in terms of lateral force and displacement to represent the building response as an equivalent SDOF system:

$$\Delta_{eq} = \frac{\sum_{i=1}^{n} m_i \cdot \Delta_i^2}{\sum_{i=1}^{n} m_i \cdot \Delta_i} \tag{13}$$

$$VB_{eq} = \frac{VB}{M_{eq}} \tag{14}$$

$$M_{eq} = \frac{\sum_{i=1}^{n} m_i \cdot \Delta_i}{\Delta_{eq}} \tag{15}$$

where Δ_{eq} is the equivalent displacement at the center of seismic force; Δ_i and VB are the floor displacement and base shear force at the peak response respectively. Once VB_{eq} and Δ_{eq} are obtained for all the accelerograms for a given building, the yield vibration period of that building is obtained using (16):

$$T_y = 2 \cdot \pi \cdot \left(\frac{\Delta_{eq}}{VB_{eq}} \right)^{0.5} \tag{16}$$

Only the pre-yield and at yielding points of the complete capacity curve are used for the computation of yield vibration period, see Figure 4 in which only the encircled points were considered for the period estimation. The vibration period was computed for all points regardless of the direction and positive/negative response of a given building. Thus, the present study represents the vibration period of the equivalent SDOF system irrespective of the direction and signs.

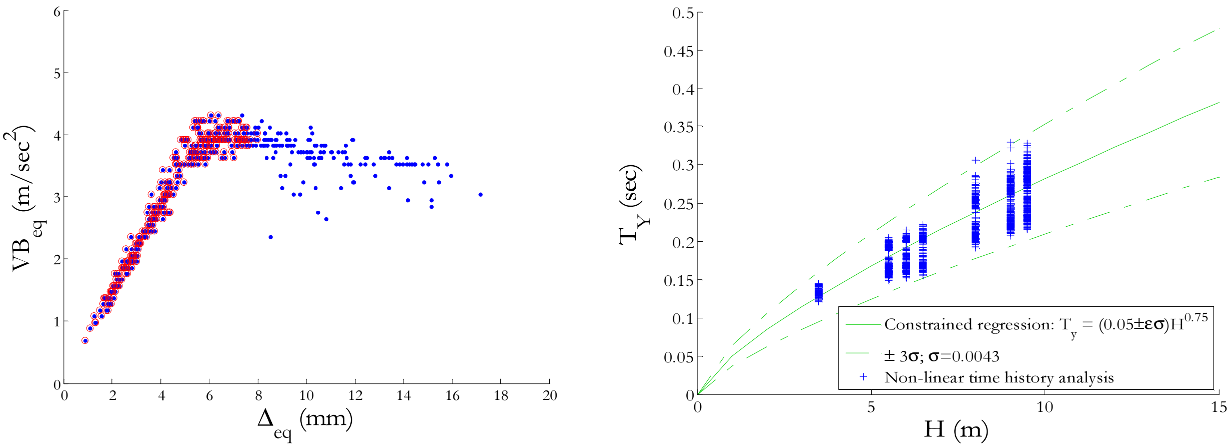

Figure 4. Left: equivalent capacity curve for three storey building, right: yield vibration period of case study buildings; nonlinear time history analysis (NLTHA) results and best fit.

The vibration period computed for all the case study buildings were plotted against their corresponding heights, in order to form an expression for the vibration period in terms of height of the building:

$$T_y = a \cdot H^b \tag{17}$$

where H is the total height of the building; a and b are the coefficients obtained through best fitting to the data. A constrained regression analysis with b=0.75 was performed to obtain the value of *a*. A mean value of 0.05 is obtained for coefficient *a* with 0.0043 logarithmic standard deviation. The constrained best fit to the data is shown in Figure 4 with ±3 standard deviations. Additionally, factor α is obtained from the identification of the slope of the post-yield branch of nonlinear response of equivalent SDOF idealization of buildings. A mean value of -0.05 is observed with a c.o.v. of 60%, which is used in the computation of post-yield vibration period of the buildings. The equivalent capacity curves derived in the present study take into account the stiffness and strength degradation due to cyclic response, higher mode effects and torsional effects of the 3D buildings under random vibration consideration.

4.2. Displacement capacities at different performance levels of the building

Simple structural mechanics concepts are used to estimate the displacement capacity of the building at the center of seismic force given the drift level in the masonry elements: piers and spandrels. Two mechanisms, column-sway and beam-sway, are considered for the global response of the building, in order to formulate the equivalent displacement capacity of the building. Column-sway mechanism is observed in masonry buildings having weak piers and strong spandrels such as buildings with reinforced concrete slab and ring beams above the spandrels. The beam-sway mechanism is mostly observed in masonry buildings having strong piers and weak spandrels; i.e. buildings usually with timber and light floors without any lintel beam and ring beam. The formula for the yield and post-yield displacement capacities at the center of seismic force for these mechanisms are presented below:

$$\Delta_y = \theta_{y-CS/BS} \cdot k_1 \cdot H \tag{18}$$

$$\Delta_{LS-CS} = \theta_{y-CS} \cdot k_1 \cdot H + \left(\theta_{LS-CS} - \theta_{y-CS}\right) \cdot k_2 \cdot H_g \tag{19}$$

$$\Delta_{LS-BS} = \theta_{y-BS} \cdot k_1 \cdot H + \left(\theta_{LS-BS} - \theta_{y-BS}\right) \cdot k_3 \cdot H \tag{20}$$

$$\theta_{BS} = \frac{\theta_{sp}}{\left(1 + \frac{D}{L_{sp}}\right)} \tag{21}$$

where $\Delta_{y-CS/BS}$ and $\theta_{y-CS/BS}$ are the yield displacement and drift value at the cracks formation in piers/spandrels, respectively; $\Delta_{LS-CS/BS}$ and $\theta_{LS-CS/BS}$ are the post-yield displacement and drift value respectively; H_g is the ground floor height; k_1, and k_3 are the coefficients to obtain the equivalent height of the SDOF system from the total building height for column-sway (CS) and beam-sway (BS) mechanism respectively; k_2 is the effective height of the ground floor pier; θ_{sp} is the limit states drift value of spandrel and θ_{BS} is the corresponding drift in the piers, which is based on the simple equilibrium consideration for horizontal coupling elements and piers.

The drift values at different performance level of walls, responding in diagonal shear mechanism, with aspect ratios of 0.66, 0.93, and 1.22 with pre-compression level of $0.14f_u$, $0.091f_u$ and $0.153f_u$ representing the common building piers are obtained from [7]. These walls were tested cyclically under the fixed-fixed boundary conditions with controlled lateral displacement for the in-plane cyclic response primarily in diagonal shear. The study reported the mean drift level of 0.088% with a c.o.v. of 8.8% at the formation of diagonal cracks in the walls, 0.22% with a c.o.v. of 35% for the maximum response of walls i.e. at the attainment of maximum shear strength in the walls, and 0.46% with a c.o.v. of 27% at the ultimate displacement capacity corresponding to the strength reduction by 20% in each of the walls. The study also reported the minimum observed values of 0.08%, 0.15%, and 0.35% at the three performance levels of cracking, maximum response, and ultimate response of the walls.

4.3. Energy dissipation of walls

The energy dissipation of the tested walls [7] is computed from the cyclic response of the walls in terms of the hysteretic damping coefficient of the walls, computed using the hysteresis area-based approach, (27):

$$\xi_{hyst} = \frac{A_h}{4 \cdot \pi \cdot A_e} \tag{27}$$

where ξ_{hyst} is the hysteretic damping coefficient; A_h is the area within one complete cycle of the hysteresis at a given level of deformation in the wall; and A_e is the area of the elastic response with the same deformation level and force observed during the cycle. The measure of the energy dissipation is reported in terms of drift attained by masonry walls. The present study attempted to represent the damping values in terms of ductility of the walls, which is well correlated with element nonlinearity than drift and is already utilized by the DBELA methodology. The displacement corresponding to the yielding, after bi-linear idealization [5], of the element is considered for the ductility computation at the post yield response. The analytical form of damping as a function of ductility is selected based on the recommendations of [3] for many other types of hysteretic response in the structural systems, as given in (28):

$$\xi = 0.05 + \frac{c \cdot (\mu - 1)}{\pi \cdot \mu} \tag{28}$$

where ξ is the equivalent viscous damping coefficient of the SDOF system and c is the coefficient computed, herein, based on the experimental results of the masonry piers. The coefficient c is obtained through nonlinear constrained regression analysis. The damping values corresponding to the ductility level less than 1.0, which are observed due to the assumed definition of yield displacement, are not used in the regression analysis. The regression analysis resulted in the value of 0.32 for c and with a dispersion of 0.0219 over the damping. The values of damping obtained experimentally and the best fit to the data using (28) with the ±1 dispersion is shown in Figure 5. The figure also shows the data actually reported [7]. In this figure H_P/D and p represents the aspect ratio and pre-compression level of the tested walls respectively.

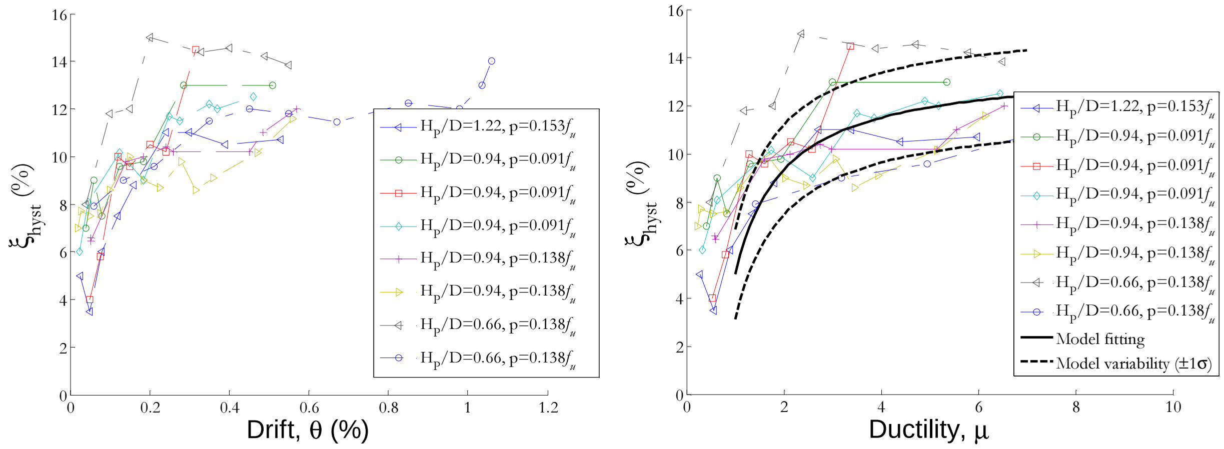

Figure 5. Hysteretic damping for URBM walls responding in shear, left: in terms of drift and right: proposed in terms of ductility.

5 CONCLUSION

The emerging concepts of performance based engineering for masonry buildings made it possible to calibrate the existing simplified methodologies developed for reinforced concrete buildings for the vulnerability assessment of masonry buildings as well. Nonlinear dynamics time history analysis of 7 case study 3D buildings is carried out for a suite of real accelerograms using a simplified formulation, in order to obtain equivalent SDOF systems of the masonry buildings taking into account the stiffness and strength degradation due to cyclic response, higher mode effects and torsional response of 3D buildings, which is used to estimate the characteristic vibration periods of masonry buildings. Deformation limit states and energy dissipation characteristics of masonry shear walls, obtained experimentally, are also presented herein. Simplified analytical formulae are proposed for the aforementioned characteristic properties of masonry buildings for the future use in the vulnerability assessment of such buildings. Future development of the proposed models can be performed using more in situ and experimental data.

ACKNOWLEDGEMENTS

Prof. Guido Magenes and Dr. Tim Sullivan of the University of Pavia Italy are thanked for their useful and encouraging discussion on the simplified dynamic analysis of masonry buildings. Dr. Mohammad Javed of NWFP University of Engineering and Technology Peshawar, Pakistan is thanked for sharing his experimental data.

REFERENCES

[1] Crowley, H.; Pinho, R. & Bommer, J.J.: A Probabilistic Displacement-Based Vulnerability Assessment Procedure for Earthquake Loss Estimation. *Bulletin of Earthquake Engineering* 2(2004) 8, 173-219.

[2] Ahmad, N.; Crowley, H.; Pinho. R. & Ali, Q.: Displacement-Based Earthquake Loss Assessment of Masonry Buildings in Mansehra City, Pakistan. *Journal of Earthquake Engineering,* 14(2010)-Special Issue.

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

[4] Magenes, G.; Bolognini, D. & Braggio, C.: *Metodi Semplificati Per l'analisi Sismica Non Lineare Di Edifici in Muratura*, GNDT, Roma, Italia 2000.

[5] Magenes, G. & Calvi, G.M.: In-Plane Seismic Response of Brick Masonry Walls, *Earthquake Engineering and Structural Dynamics* 26 (1997) 11, 1091-1112.

[6] F. McKenna, G.L. Fenves & M.H. Scott, 'Open system for earthquake engineering simulation: version2.1.0', *URL* http://opensees.berkeley.edu 2008.

[7] 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.

Note: to cite; Ahmad, N., Crowley, H., Pinho, R., Ali, Q., [2010] "Simplified formulae for the displacement capacity, energy dissipation, and characteristic vibration period of brick masonry buildings", *Proceedings of the International Masonry Society* 11(2), pp. 1385-1394.