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Pushover Analysis of Masonry Buildings

Kamal Sonekar[1], S. V. Bakre[2]
[1]*Post-graduate student, VNIT, Nagpur*
[1]*Tel. no- 09765337536, E-mail-sonekarkamal@gmail.com*
[2]*Associate Professor,*
VNIT, Nagpur
[2]*E-mail-svbakre@yahoo.com*

Abstract : *Masonry is a common material for building construction but is known for its seismic vulnerability. The paper presents a comparative study on non-linear behavior of masonry frame structures when subjected to earthquake excitation under different lateral loading pattern. Equivalent Frame Model (EFM) is being used for modeling the non-linear behavior of masonry by providing flexural and shear hinges in the model. The flexural hinges are defined based on equations derived through experimental study on unreinforced masonry (URM) walls. The shear hinges are defined using equations obtained through regression analysis of bi-linearized pushover curves of the URM walls modeled as finite element macro model in ANSYS software. Analysis is done in SAP2000 software and useful conclusions on strength and inelastic properties are drawn.*

Then seismic performance and vulnerability of a masonry building is studied using the same modeling technique used for frame structures. Four quality levels of masonry i.e. slight, moderate, extensive and complete were considered to represent variability in seismic performance of building and finally fragility curves were obtained based on spectral displacements and damage probability. It is observed that the building have more probability for moderate damage.

1. INTRODUCTION

URM construction is common construction practice in many places of the world. It has gained popularity mainly due to its low cost, widespread geographical availability, thermal insulation, protection from fire, durability, low maintenance cost and it is easy to construct. Normally, masonry is designed for vertical loads since it has good compressive strength. Hence, the structures will behave well as long as the loads are vertical but when horizontal inertial earthquake forces act, they start to develop shear and flexural stresses. Since not much technological development and research is done in this area and due to little skill required, masonry construction is often done without any engineering knowledge. This makes URM construction vulnerable to earthquakes.

As a result, RCC and steel are replacing masonry as a construction The existing URM construction possesses a risk during earthquakes. Therefore, for performance based earthquake engineering concepts need for non-linear static analyses arises. In recent years non-linear methodologies like Pushover Analysis is being used for retrofitting and rehabilitating existing buildings. Pushover analysis is an approximate analysis method in which the building model is subjected to a predefined load pattern and the loads are increased monotonically until some members yield. The structure is modified for reduced stiffness of the yielded members and the loads are again increased until a control displacement is reached or the structure becomes unstable.

For Pushover analysis non-linear hinges are required to be inserted in the model. The non-linear properties of these hinges are based on the failure mechanisms occurring in masonry. The various failure mechanisms are,[1]

Rocking- It is a flexure dominated failure in which flexural cracks are developed at the top and bottom of a wall.

Diagonal shear- It characterized by either is horizontal cracks along bed joints or stair stepped cracks along bed and head joints.

Diagonal tension- Shear dominated failure with diagonal cracking in the middle of wall.

Toe Crushing- It is characterized by crushing of masonry at maximum compressive zone which is generally located at the bottom end of the wall.

2. NON-LINEAR MODELING OF MASONRY

2.1. Equivalent Frame Model (EFM)

Equivalent frame method is a simple and effective method to carry out non-linear analysis of URM structures. Since homogeneous and isotropic material idealization is being made, less amount of data and hence least experimental study is required to determine the mechanical properties of the materials. In this method, the element is modeled as an equivalent frame having same dimensions of an actual element. The structure is modeled as an assemblage of horizontal and vertical members called spandrels and piers respectively. The non-linear behavior of the elements is described by providing non-linear hinges whose force displacement properties are usually defined from experimental results.

The effective heights of piers and spandrels are determined based on the approach given by Dolce M. (1989)[2] as shown in Figure 1. Dolce M. ha given criteria for defining Rigid offsets at the ends of piers and spandrels so as to define a connection between them.

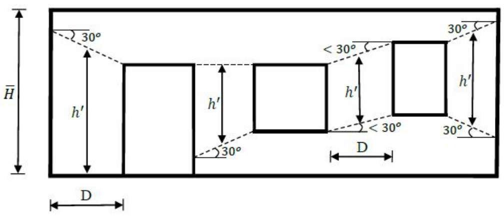


Figure 1. Effective height determination offered by Dolce M. (1989)

2.2. Definition of non-linear hinges

2.2.1. Shear-Displacement Hinges

A. Aldemir (2010)[3], carried out pushover analysis on more than 300 finite element models of URM walls using ANSYS software. Finally, he derived pushover curves, bi-linearized them and derived equations for Ultimate force, Yield force and Ultimate displacements as given below,

$$F_y , F_u = C_1 \times p^{C2} \times f_m^{C3} \times e^{C4\lambda} \times h \times t \quad \text{......(1)}$$

$$\delta_u = C_1 \times p^{C2} \times e^{C3fm} \times \lambda^{C4} \times h \times t \quad \text{.......(2)}$$

where, p, overburden pressure on wall; f_m, compressive strength of masonry wall; λ, aspect ratio of wall; h, height of cross section; t, thickness of wall. The coefficients used in above equations are given in Table 1. The non-linear force-displacement behavior of hinges is shown in Figure 2. Within elastic limit, Lateral force is related to lateral displacement by lateral elastic stiffness, Ke, which is determined using equation (3). While in the non-linear range, ultimate shear and ultimate displacement values are to be found out using equations (1) and (2) respectively.

Table 1. Coefficients used in Eq. (1) and (2)

Coefficients	Yield Lateral Load	Ultimate Lateral Load Capacity	Ultimate Lateral Displacement Capacity
C1	353.2	352.2	2.385
C2	0.604	0.498	-0.540
C3	0.414	0.501	0.319
C4	-0.931	-0.856	1.414

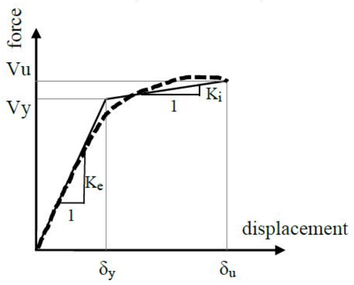


Figure 2. Bilinear idealization of force-displacement relationship for masonry walls

$$k_e = \left[\frac{5\beta E t}{60(\lambda)^3 + 12\beta(1+\nu)\lambda} \right] \quad \text{.........(3)}$$

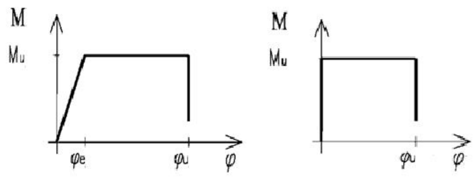
where, ke, total lateral stiffness of URM wall; h, height of a wall or a pier; β, boundary condition parameter (3 for cantilever wall and 12 for fixed wall); E, modulus of elasticity in compression; t, thickness of a wall; ν, poison's ratio; λ, Aspect ratio of a wall or a pier.

2.2.2. Moment-Rotation Hinges

Moment-rotation hinges are defined using equation (4) given by Magenes and Calvi (1996)[4]. The piers are modeled as elasto-plastic with final brittle failure (Figure 3(a)). A rigid-perfectly plastic behavior is assumed for the hinge (Figure 3(b)).

$$Mu = \frac{\sigma_0 D^2 t}{2}\left(1 - \frac{\sigma_0}{k f_d}\right) \quad \text{.......... (4)}$$

where, σ_0, mean vertical stress; D, width of pier; t, thickness of pier; k, the coefficient taking into account the vertical stress distribution at the compressed toe (a common assumption is an equivalent rectangular stress block with *k*=0.85); f_d , design compression strength.



(a) (b)

Figure 3 (a) and (b). Behavior assumed, respectively, for the entire pier and the correspondent plastic hinge (Pasticier, 2007[5])

The position of hinges in the model depends on the possible failure mechanism occurring location. But, hinges can be inserted anywhere in the frame element[6]. Here, one shear hinge is provided in the middle while two flexural hinges are provided at the ends of an element.

Other parameters needed as input in SAP2000 for defining non-linear properties of the hinges are taken from FEMA 356[7].

3. PUSHOVER ANALYSIS OF FRAME STRUCTURE

A two storey two bay URM frame as shown in Figure 4 is considered. The basic mechanical properties of masonry are Young's modulus, E_m, 1700 MPa; Poisson's ratio, ν, 0.2; Unit wt. of masonry, 17 kN/m3; Mean compressive strength of masonry, 6.2 MPa.

An initial linear analysis of model is done under dead loads to determine axial load in each pier and vertical pressure coming on them. The cross section properties of each pier are determined. The hinge properties found out using Eq. (1), (2) and (3) are given in Table 2. EFM with hinges assigned to it is shown in Figure 5.

The pushover curves obtained are as shown in Figure 5. Comparison is made based on the lateral load pattern.

Uniform- Load applied at each node is proportional to mass tributary to that node.

Mode- Load at each node is proportional to displacement in first mode times the mass tributary to that node.

Parabolic- Load at each node is proportional to the load pattern as given in IS 1893(part 1):2002[8].

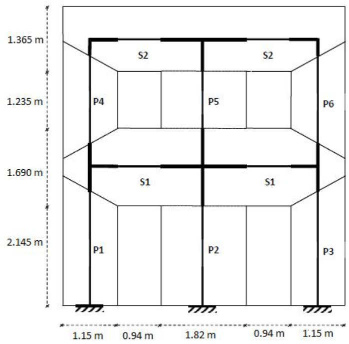


Figure 4. 2 storey, 2 bay URM frame

Table 2. Flexure and shear hinge properties

Pier	F_y(kN)	F_u(kN)	δ_u(mm)	M_u(kNm)
P1	15.35	23.603	25.98	57.032
P2	64.861	91.437	16.41	162.72
P3	15.35	23.603	25.98	57.032
P4	15.084	24.311	27.53	25.623
P5	64.681	95.35	10.734	74.942
P6	15.084	24.311	27.53	25.623

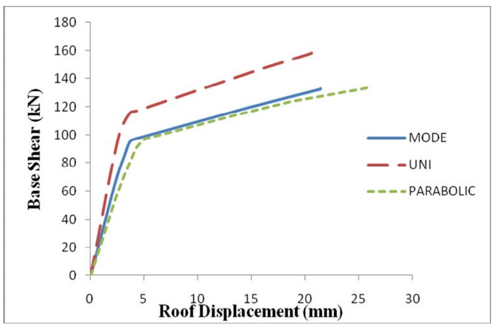


Figure 5. Pushover curves for 2 bay 2 storey URM frame

4. SEISMIC ASSESSMENT OF EXISTING URM BUILDING

The plan of a two storey URM building is shown in Figure 6. Compressive strength of masonry is 5 MPa; E_m, 2000 MPa; ν, 0.2; unit wt. of concrete is 25 kN/m^3 and masonry is 18 kN/m^3. [2]

The non-linear hinge properties for all the piers are determined similarly using equations (1),(2) and (3). The building is subjected to a parabolic load pattern as per IS 1893(part 1):2002 in both longitudinal and transverse directions. The pushover curves obtained in X and Y directions are converted to ADRS (Acceleration Displacement Response Spectrum) format as per the criteria given in FEMA 356[7] for seismic performance assessment.

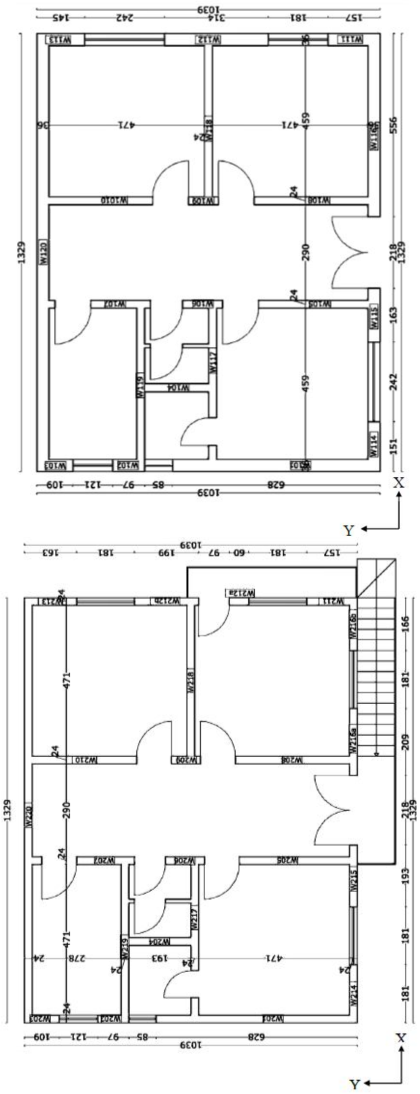


Figure 6. First and Second floor plan of the building

The capacity curves are shown in Figure 7. The capacity curves are bi-linearized to obtain yield and ultimate spectral displacement values.

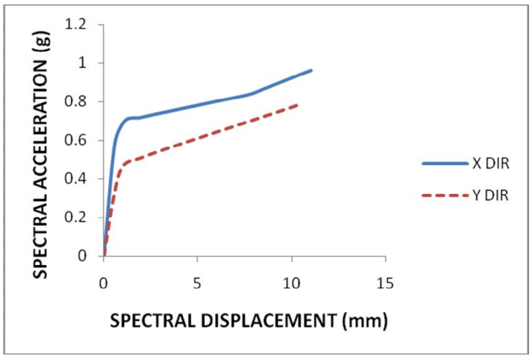


Figure 7. Capacity spectra for the considered building

Damage states define the physical condition of a building subjected to earthquake. Four damage states are described in (HAZUS-MR1)[9] i.e. slight, moderate, extensive and complete. Fragility curves are plotted in both longitudinal and transverse direction of building for all four damage states. Fragility curves gives the probability of reaching or exceeding a particular damage state, as a function of severity of seismic ground motion to which a building is subjected. Generally, spectral displacement is used to express the severity of ground motion. The probability of exceedance of a given damage state and are lognormal distributions, defined as, [9]

$$P[Gr \geq Gr_i / S_d] = \phi\left[\frac{1}{\beta_{ds}} \ln\left(\frac{S_d}{S_{d,ds}}\right)\right]$$

$$\beta_{ds} = \sqrt{\left[CONV\left[\beta_C, \beta_D\right]\right]^2 + \left(\beta_{T,ds}\right)^2}$$

where, β_{ds} - Standard deviation of the natural logarithm of spectral displacement for damage state, ds, βc- Lognormal standard deviation used to describe variability in capacity curve, β_D- Lognormal standard deviation used to describe variability in demand curve, $\beta_{T,ds}$- Lognormal standard deviation used to describe variability in threshold of damage states, $S_{d,ds}$ - Median value of spectral displacement at which the building reaches the threshold of damage state, ds. Φ - Standard normal cumulative distribution function. The values of $S_{d,ds}$ are taken from Kappos et. al.[10] as given in Table 3.

HAZUS-MR1 has provided pre- computed values of β_{ds} to avoid complex numeric calculations. The variability values are shown in Table 4 for the parameters assumed in the study.

Table 3. Damage state thresholds (Kappos et. al)

Damage state	Damage state label	Spectral Displacement
Gr0	None	<0.7 Sd_y
Gr1	Slight	0.85 Sd_y
Gr2	Moderate	1.5 Sd_y
Gr3	Extensive	2 Sd_y + (Sd_u – 0.7Sd_y)/2
Gr4	Complete	0.85 Sd_u
Gr5	Collapse	Sd_u

Table 4. Variability considerations

Damage state	Kappa(k) factor	Damage variability ($\beta_{T,ds}$)	Capacity curve variability (βc)	Total variability (βds)
SLIGHT	Minor degradation (0.9)	Moderate (0.4)	moderate (0.3)	0.8
MODERATE	Major degradation (0.5)	moderate (0.4)	moderate (0.3)	0.95
EXTENSIVE	Extreme degradation (0.1)	moderate (0.4)	moderate (0.3)	1.05
COMPLETE	Extreme degradation (0.1)	moderate (0.4)	moderate (0.3)	1.05

5. RESULTS

In case of URM frame, higher strength estimates are obtained for mode load case while uniform and parabolic load cases are found to be nearly equivalent. Noticing the formation of hinges at the final step of pushover analysis, bottom interior pier is seen to have reached ultimate capacity for all the three load cases. Shear yielding was seen first to occur.

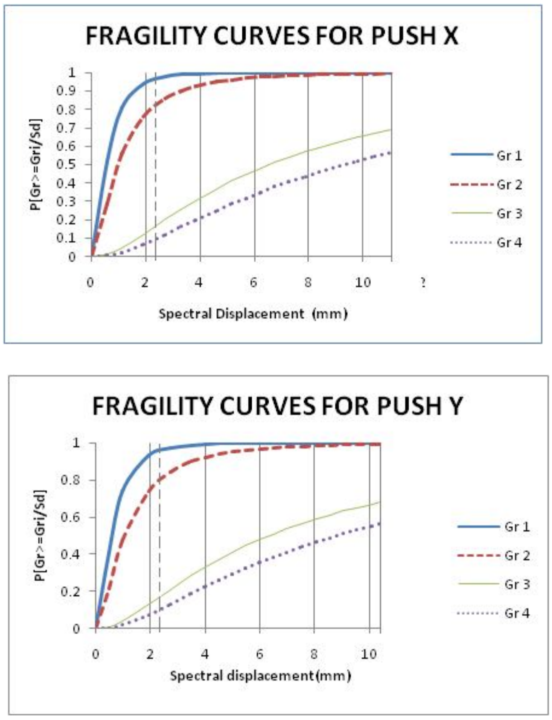


Figure 8. Fragility curves for considered building

The building is found to be capable of carrying 73.2% of its total weight in X direction and 60% of its total weight in Y direction. Also, ultimate total drift is 2% in X direction and 1.86% in Y direction. Spectral displacement in strong direction X is found to be ($1.008\mathrm{x}10^{-3}$)m which is less than in weak Y direction having spectral displacement ($2.805\mathrm{x}10^{-3}$ m) stating that the building displaces less in strong direction as compared to weak direction. But spectral acceleration for X direction is found to be 0.678 kN/m^2 while in Y direction it is 0.537 kN/m^2 stating that the building accelerates more in strong direction expecting more damage to non structural components and components sensitive to acceleration. The probability of damage is found to be more i.e. around 66 % and around 63 % for Moderate damage state for 2.3 mm spectral displacement in both X and Y directions respectively.

6. CONCLUSIONS

The non-linear behavior of URM frame was studied by carrying out pushover analysis under three different lateral loadings and then seismic vulnerability of an existing URM building was done. On the basic of the results obtained the following major conclusions can be drawn,

1. EFM is simple, approximate and economic method for modeling masonry as compared to much complicated and tedious FE macro modeling.
2. As far as maximum developed strength is considered, higher strength estimates are obtained for uniform load pattern along the height of the structure while mode and parabolic lateral load patterns are found to be always equivalent (i.e. around 15% higher).
3. Shear failure is seen to be main criteria for failure of URM frame structures.
4. Spectral displacement is found to be less in strong direction as compared to weak direction (i.e. around 64 % less), stating, stronger and stiffer construction displaces less than weaker and more flexible construction for the same level of spectral demand, and less damage is expected to the structural system and nonstructural components sensitive to drift.
5. Spectral acceleration is found to be more in strong direction as compared to weak direction (i.e. around 27% more), stating, stronger and stiffer construction will shake at higher acceleration levels, and more damage is expected to nonstructural components and contents sensitive to acceleration.

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