Seismic Fragility Curves for High-Rise Masonry Infilled Reinforced Concrete Building

Phadnis P.P.[1]

[1]*Assistant Professor, Department of Technology, Shivaji University, Kolhapur, 416004, Maharashtra, India*

---***---

Abstract - *The present work aims to study the effect of various configurations of brick masonry infill (BMI) in reinforced concrete (RC) building on seismic fragility curves using capacity spectrum method. Spectral displacement-based fragility curves and damage probability matrix under various damage states are developed for highrise RC frames. The comparative fragility curves for buildings with various configurations of BMI are presented. It is concluded that the BMI frame have shown superior performance during earthquake in all modes of different damage states.*

***Key Words*: brick masonry infill, equivalent strut, capacity spectrum, fragility curves, damage probability**

1. INTRODUCTION

Building seismic performance and vulnerability analysis with its probabilistic nature is currently of structural designer's interest. Precise building vulnerability may be determined by developing fragility curves. Fragility curves describe the probability of damage to the building. Building fragility curves are lognormal functions that describe the probability of reaching or exceeding damage states at given median estimates of spectral response for example spectral displacement. These curves take into account the variability and uncertainty associated with capacity spectrum characteristics, damage levels and ground shaking. Many researchers in the past studied and proposed empirical and analytical methods to develop fragility curves for the potential risk management. Ray Kai Leung Su, et al. [1] proposed a simplified method to develop fragility curves for low rise brick masonry infill based on coefficient method. In FEMAP-58-1 [2] report, seismic performance assessment of buildings, and its companion volumes, together describes the consequence methodology as well as the development of basic building information, response quantities, fragilities and resulting data used as inputs to the methodology. Kursat Kinali, et al. [3] paper described an approach to the building fragility assessment component of consequence based design, and illustrated this approach for steel frames typical of regions of low-to-moderate seismicity in the Central and Eastern United States. Astriana L. et al. [4] developed spectral acceleration based fragility curves according to HAZUS-MH MR5 and ATC 40 from capacity spectrum method.

In this paper, the deterministic non-linear behavior and capacity of the structures is obtained from non-linear pushover analysis. The median spectral displacements for various damage states are determined from capacity spectrum of buildings with various configurations of BMI. Damage states are determined according to Barbat, et al. [5] and HAZUS technical manual [6].

2. METHODOLOGY TO DEVELOP FRAGILITY CURVE

2.1 Pushover Analysis

Non-linear static (pushover) analysis are initially performed to assess seismic performance of the structures and afterwards used also for fragility analysis. In pushover analysis set of load is incrementally applied to determine force–displacement relationship in terms of capacity curve.

In comparison with non-linear time history analysis, simplicity, ease to perform and lesser consumption of computational time during analysis are advantages of pushover analysis and its results are quite intuitive. However, to interpret accurate results, the analyst should have solid background of knowledge and understand the basic assumptions involved. But its accuracy decreases when the higher modes become predominant in structure.

2.2 Capacity Spectrum

Capacity curve is a plot of lateral force Vs roof displacement of structure derived from incremental non-linear static analysis. It is converted into capacity spectrum as per ATC40 prescribed method. Capacity spectrum is a plot of spectral displacement (S_d) against spectral acceleration (S_a). The demand imposed on the structure i.e. design response spectrum is a plot of spectral acceleration (S_a) and time period (T). Both capacity spectrum and demand response spectrum are plotted on graph in ADRS format.

2.3 Damage State Development

For each capacity spectrum of the structures considered, the damage state, Barbat, 2008 and HAZUS, 2003 criteria are utilized in this investigation. Barbat four levels of damage are stated in Table 3.2.

Table 1: Barbat Damage state thresholds [5]

Damage State	Median spectral displacement ($S_{d,ds}$)
Slight	$S_{d,S} = 0.7\ S_{d,y}$
Moderate	$S_{d,M} = S_{d,y}$
Extensive	$S_{d,E} = S_{d,y} + 0.25\ (S_{d,u}-S_{d,y})$
Complete	$S_{d,C} = S_{d,u}$

According to HAZUS 2003, four levels of damage are defined as enlisted in Table 3.2.

Table 2: HAZUS Damage state thresholds [6]

Damage State	Median spectral displacement ($S_{d,ds}$)
Slight	$S_{d,S} = S_{d,y}$ (First yield)
Moderate	$S_{d,M} = 1.5S_{d,y}$
Extensive	$S_{d,E} = 0.5(S_{d,M} + S_{d,C})$
Complete	$S_{d,C}$ = Average median value from capacity spectrum of building near collapse state

2.4 Fragility Function

Due to random nature of earthquakes, the structural response at given seismic loading to be best determined by probabilistic approach rather than deterministic. Hence, for performance based earthquake engineering (PBEE), the distribution of structural response (Engineering Demand parameter, EDP i.e. maximum inter-story drift ratio) is correlated probabilistically with Intensity measure (IM, i.e. spectral displacement, spectral acceleration, peak ground acceleration, etc.). This distribution is formulated by related probability function of EDP for given IM i.e. P(EDP|IM). For a given peak spectral displacement S_d', at given median displacement $S_{d,ds}$ and standard deviation β_{ds} for particular damage state ds, Ø is standard normal distribution function, the conditional probability (P) of being in or exceeding defined by

$$P\left[\frac{ds}{S_d}\right] = \emptyset\left[\frac{1}{\beta_{ds}} ln\left(\frac{S_d'}{S_{d,ds}}\right)\right] \quad (1)$$

Discrete damage probabilities of particular damage states are defined as:

Probability of complete damage,

$P[C] = P[C|S_d]$

Probability of extensive damage,

$P[E] = P[E|S_d] - P[C|S_d]$

Probability of moderate damage,

$P[M] = P[M|S_d] - P[E|S_d]$

Probability of slight damage,

$P[S] = P[S|S_d] - P[M|S_d]$

Probability of no damage,

$P[N] = 1 - P[S|S_d]$ (2)

2.5 Uncertainty Response Parameter

$$\beta_{Sds} = \sqrt{\left[conv(\beta_c, \beta_D, S_{d,Sds})\right]^2 + {\beta_{M,Sds}}^2} \quad (3)$$

where,

β_{Sds} is lognormal standard deviation that describes total variability for structural damage state, ds

β_c is the lognormal standard deviation parameter that describes the variability of the capacity curve,

β_D is the lognormal standard deviation parameter that describes the variability of the demand spectrum,

$\beta_{M,Sds}$ is the lognormal standard deviation parameter that describes the uncertainty in the estimate of the median value of the threshold of structural damage state, ds,

CONV The function "CONV" implies a complex process of convolving probability distributions of the demand spectrum and the capacity curve, respectively.

3. NONLINEAR MODELING OF BRICK INFILL

The analytical modeling of infill frames is complex in nature, because these structures reveal highly nonlinear inelastic behavior, resulting from the interaction of the masonry infill panel and the surrounding frame. Many researchers have been proposed various modeling approach. Hemant B. Kaushik, et al. [8] compared single-strut model, 3-strut model, and finite element models and the effectiveness of these three models in seismic analysis is explained. Das D. and Murthy C.V.R [9] modeled Brick infill panel as equivalent braced frame according to Mainstone. Davis R. et al. [10] were considered infill using equivalent-strut approach suggested by Smith Static. Sommoila D.M. [11] modeled single strut and three diagonal struts and suggested that single strut model is better to be used in analysis regarding general behavior of infill frames. Jamnekar V.P. and Durge P.V. [12] modeled

BMI using equivalent diagonal strut approach using Main-stone theory.

For infill wall located in a lateral load resisting frame the stiffness and strength contribution of the infill are considered by modeling the infill as an equivalent strut approach given by FEMA- 356 [7] as below:

Width of strut is given by Width of strut is given by

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf} \quad (4)$$

$$\lambda_1 = \left[\frac{E_m t \sin(2\theta)}{4E_{fe} I_{col} h_{inf}}\right]^{1/4} \quad (5)$$

where,

h_{col} = Column height between centre lines of beams

h_{inf} = Height of infill panel

E_{fe} = Expected modulus of elasticity of frame material

E_m = Expected modulus of elasticity of infill material

= $550 \times f_m$

I_{col} = Moment of inertia of column

r_{inf} = Diagonal length of infill panel

t = Thickness of infill panel and equivalent strut

θ = Diagonal angle

f_m = Compressive strength of masonry

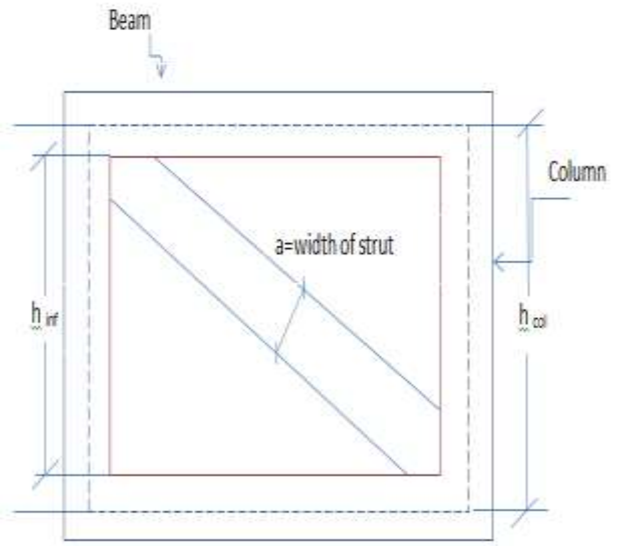


Fig -1: Equivalent width of strut

In this study, four different models of eleven storey building symmetrical in plan are considered. Buildings are modeled using 40% masonry infill, but arranging them in different configuration as shown in fig. 2. The building has four bays in x-direction and y-directions with plan dimensions 20m×16m and storey height of 3.0m each in all floors. Size of beam is 450mm×600mm and size of column is 750mm×750mm for bottom four stories and 450mm×600mm for upper stories. The columns are assumed to be fixed at the ground level. Depth of slab is considered as 120mm. Weight of floor finishes is 1.5kN/m^2. Imposed load is considered as 2kN/m^2 on roof and 4kN/m^2 on floor. Slab loads have been distributed to frame elements according to yield line pattern.

M20 grade of concrete is used with modulus of elasticity 22360MPa, Fe415 grade of steel is used with yield strength of 415MPa with elastic modulus 2×10^{5}MPa and unit weight of brick masonry is 20kN/m^2 with modulus of elasticity 2035MPa. Site located in Indian Seismic zone V.

Following four different models are investigated in study.

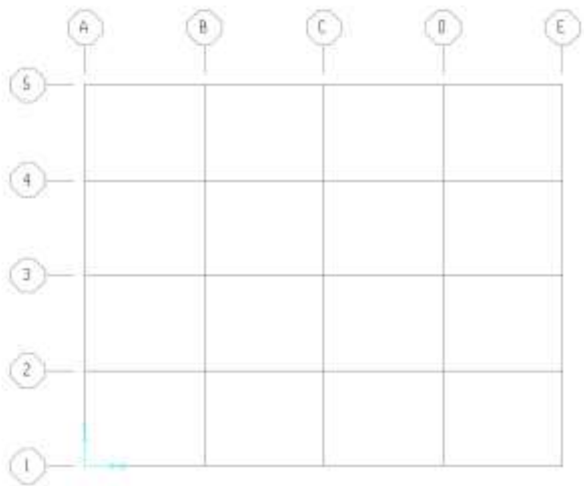
Model I : Bare frame

Model II : Masonry infill are arranged in outer periphery

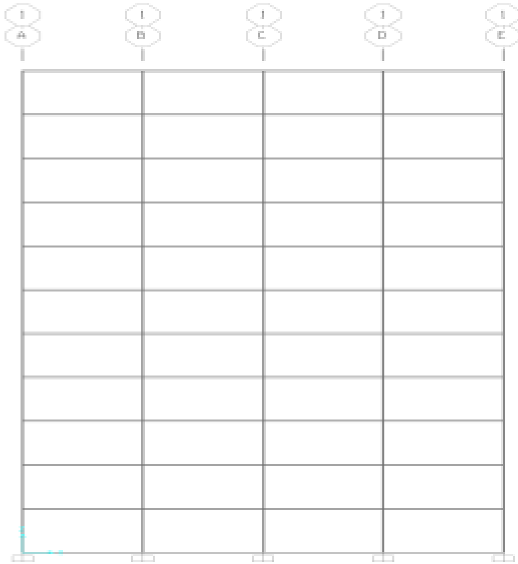
Model III : Masonry infill are arranged in outer periphery with soft storey

Model IV : Masonry infill are arranged in lift core

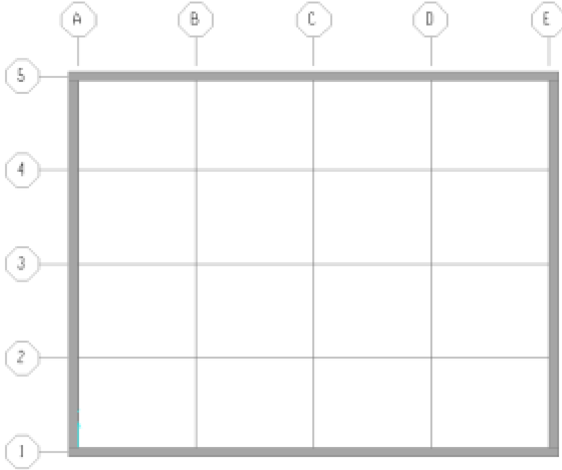
Nonlinear Static Procedure, a model is subjected to gravity analysis (DL+0.5LL) and simultaneously displaced using preselected lateral load pattern until roof displacement reaches to a target displacement, and resulting internal deformations and forces are determined.



Plan

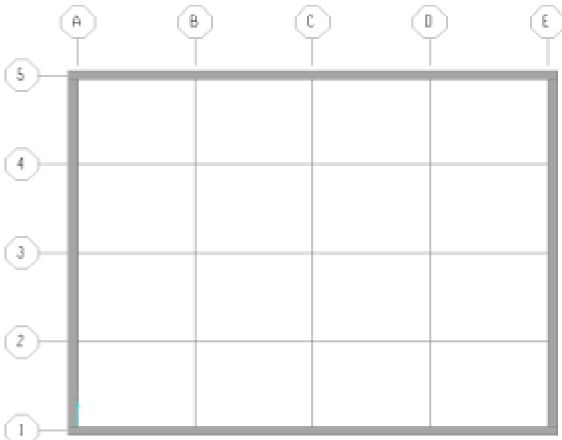


Sectional elevation

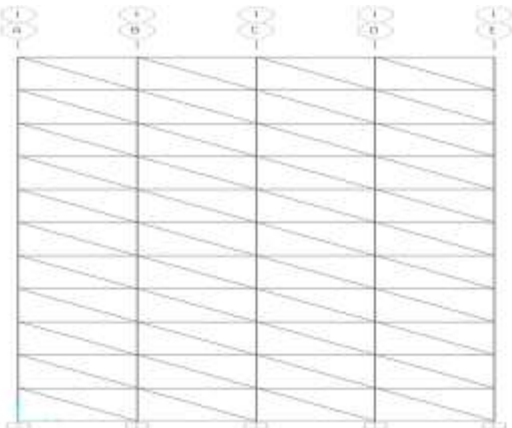


Plan

Model I

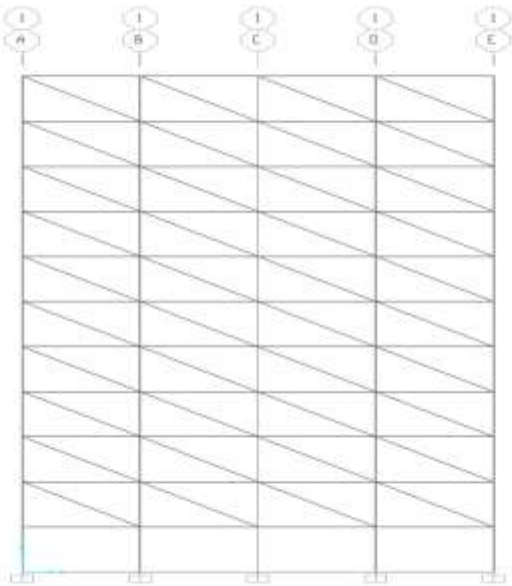


Plan



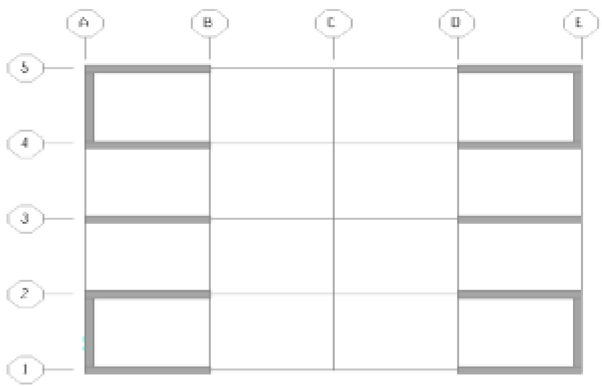
Sectional elevation

Model II

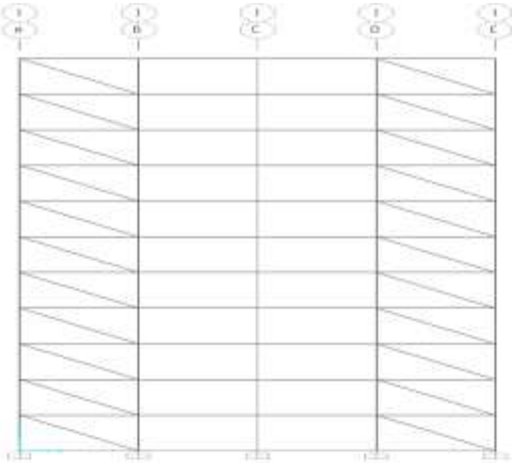


Sectional elevation

Model III



Plan



Sectional elevation

Model IV

Fig -2: Plan and Elevation of Eleven Story Reinforced Concrete Buildings

5. NONLINEAR MODELING OF FRAMES

Mander's confined model for concrete, Park's model for rebar and simple model for structural steel was used to incorporate nonlinear behavior of materials. The associated material nonlinearity in the frame elements can be modeled by assigning user defined or default plastic hinges. Mehmet Inel, et al. [11] shown that user defined plastic hinges give better results for nonlinear analysis of RC frames. The present study is focused on development of fragility curves which is probabilistic study, so the difference in the resulting fragility curves derived from user defined or default plastic hinges are not significant, hence default hinge defined by FEMA 356 [12] are used. For beams M3 hinge and for the columns, PMM hinge interaction is assigned. To model the finite size of joints, two stiff zones have been considered at the ends of the elements. The nonlinearity has been concentrated at the ends of the joints and elements by introduction of plastic hinges. The beam portion has been considered as elastic between two hinges at the ends. Axial P hinge is assigned to strut at centre.

6. RESULTS AND DISCUSSIONS

6.1 Fundamental natural period

Fundamental natural period are tabulated in table 3.

Table 3: Fundamental Natural Time period (sec) of various models

Model No.	Model I	Model II	Model III	Model IV
SAP2000	0.6883	0.6417	0.6447	0.6725

Fundamental Natural Time periods for partially infill RC frames are observed to be decreased by 6.77%, 6.33% and 2.30% in comparison with bare frame for first mode. Introduction of infill panel in the reinforced concrete frame reduces time period w.r.t. bare frame indicating increase in stiffness.

6.2 Capacity Spectrum

The performance point is intersection of capacity spectrum (CS) and demand spectrum (DS) in Acceleration Displacement Response Spectrum (ADRS) format as per ATC-40. Prakash V. [15] mapped the soil sites given in ATC-40, FEMA 356 to those given in IS: 1893-2016[16] according to shear wave velocity and standard penetration resistance is between 10 to 30, seismic coefficient C_a and C_v, are selected as user defined values of Ca is used as 0.44 and C_v is used as 0.64 according to ATC40 for soil profile type D to update capacity spectrum curves. Performance points are obtained from capacity spectrum method are shown in Table 4.

Table 4: Performance Point

Model No.	F(kN)	Δ (mm)	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total
Model I	4551.63	121.79	1104	208	110	0	0	0	8	0	1430
Model II	5571.18	80.42	1184	350	66	6	0	0	0	0	1606
Model III	5219.07	86.10	1243	283	60	0	0	4	0	0	1590
Model IV	5522.38	79.77	1147	369	53	28	0	2	7	0	1606

Base shear at performance point for Model II, III and IV are observed to be decreased by 22.39%, 14.66%, 21.33% as compared to Model I. Displacement at performance point for Model II, III and IV are observed lesser by amount 33.97%, 29.30%, 34.50% than Model I.

Performance points in ADRS format are shown in chart 1.

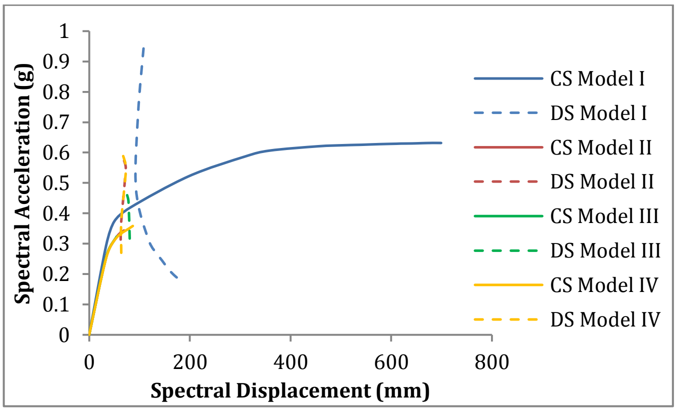


Chart -1: Capacity Spectrum

From chart 1, it is illustrated that, performance point for buildings with infill frame lies well within elastic limit in comparison with bare frame model (without infill).

6.3 Fragility Curves

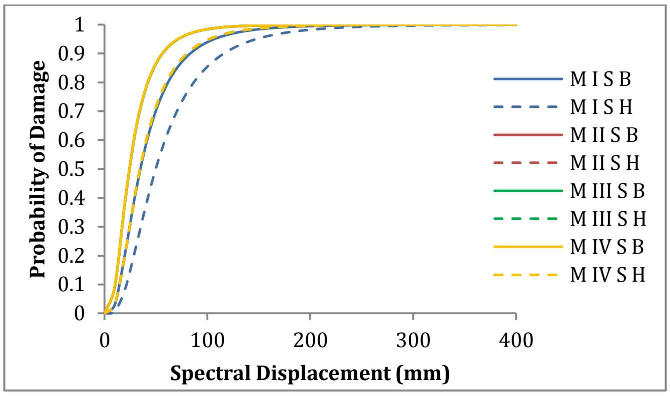
In this case, the classification of the group of building is stated in the Table "Model Building Types" (Table 5.1) of the HAZUS document, where the type C2H for High-rise Moment Resisting Framed buildings with equal to eleven floors is selected. The standard deviations are specified in the Table "Structural Fragility Curve Parameters" (Table 5.9 of the HAZUS manual). This Table is divided in four categories, giving four different threshold values of the variability according to the seismic design level: High-Code, Moderate-Code, Low-Code and Pre-Code buildings. Spectral displacement is used as input parameter. For newly proposed construction (Section 5.4.1, HAZUS), High-code threshold values are used to define the damage state limit values. The damage thresholds and variability is given in Table 5.

Table 5 Damage state thresholds and variability

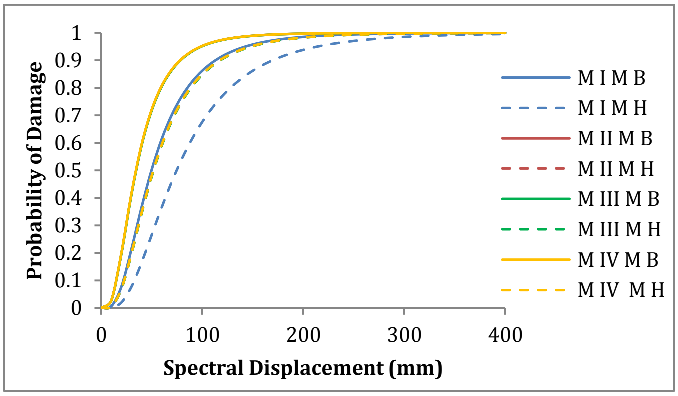
Model No.	Fragility Parameters	Slight	Moderate	Extensive	Complete
Model I	$S_{d,ds}$ (mm) (Barbat)	34.8705	49.815	212.1433	699.128
	$S_{d,ds}$ (mm) (HAZUS)	49.815	74.7225	374.4715	699.128
	β_{ds}	0.66	0.64	0.67	0.78
Model II	$S_{d,ds}$ (mm) (Barbat)	24.1325	34.475	43.166	69.239
	$S_{d,ds}$ (mm) (HAZUS)	34.475	51.7125	51.857	69.239
	β_{ds}	0.66	0.64	0.67	0.78
Model III	$S_{d,ds}$ (mm) (Barbat)	24.2228	34.604	39.5985	54.582
	$S_{d,ds}$ (mm) (HAZUS)	34.604	51.906	44.593	54.582
	β_{ds}	0.66	0.64	0.67	0.78
Model IV	$S_{d,ds}$ (mm) (Barbat)	24.0884	34.412	47.483	86.696
	$S_{d,ds}$ (mm) (HAZUS)	34.412	51.618	60.554	86.696
	β_{ds}	0.66	0.64	0.67	0.78

Variability's of all models are same because influence of masonry infill is not permitted by current seismic codes.

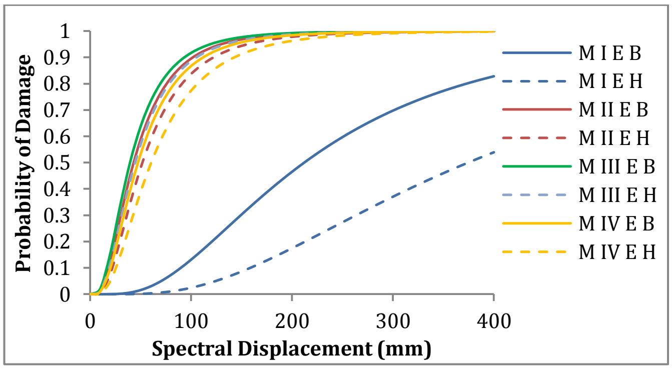
Fragility curves are determined for four different models as per Barbat and HAZUS criteria as per Equ. 1 are presented in Chart 2.



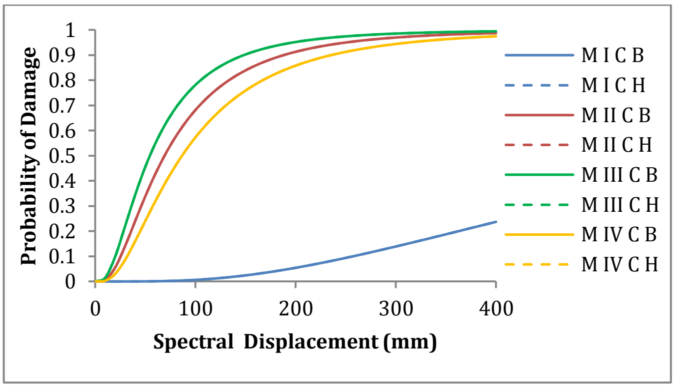
a) Slight Damage State



b) Moderate Damage State



c) Extensive Damage State



d) Complete Damage State

Chart -2: Fragility curves

Comparison of fragility curves for Barbat, 2008 and HAZUS, 2003 damage illustrates that, the HAZUS damage state give lower probability of failure for slight, moderate and extensive damage state. While for complete damage state, no difference is observed between two because of complete damage state is same for both criteria.

Damage probability for different damage states are calculated with respect to spectral displacement of 150 mm as per Equ. 2 is as follows:

Table 6 Damage Probability (%) at S_d =150mm

Model No.	No Damage	Slight	Moderate	Extensive	Complete
Model I, Barbat	1.5953	2.6548	65.5044	27.8156	2.4299
Model I, HAZUS	4.7442	9.0671	77.5838	6.175	2.4299
Model II, Barbat	0.2817	0.7978	2.0713	12.9306	83.9186
Model II, HAZUS	1.2944	3.5116	0.839	10.4364	83.9186
Model III, Barbat	0.2867	0.8095	1.2453	7.4061	90.2524
Model III, HAZUS	1.3134	3.5512	1.3540	6.237	90.2524
Model IV, Barbat	0.2794	0.792	3.2293	19.8066	75.8927
Model IV, HAZUS	1.2852	3.4925	4.011	15.3186	75.8927

Chart 3 represents discrete damage probabilities in graphical format.

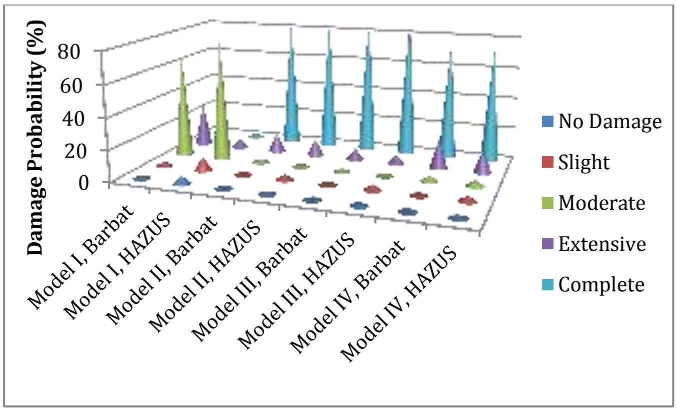


Chart -3: Discrete Damage Probabilities at S_d =150mm

Damage probabilities of slight, moderate and extensive damage state of Model II are decreased by 69.95%, 96.84% and 53.51%. For Model III moderate and extensive probabilities of damage are decreased by 98.72% and 62.48% and Model IV, slight and moderate and extensive probabilities of damage are decreased by 70.17%, 95.07% and 28.79% for Barbat criteria w.r.t. Model I. Damage probabilities of slight and moderate damage state of Model II are decreased by 61.27% and 98.92% for Model III decreased by 91.07% and 98.39% and Model IV decreased by 61.48% and 94.83% for HAZUS criteria w.r.t. Model I.

Damage probabilities of moderate and extensive damage state of Model II, III and IV with partial infill has least damage probability for three initial damage states. Model II with infill arranged at periphery provides the least damage probability because arrangement of symmetrical infill at outer extremities results into reduction in twist during earthquake. This influence that, infill affects stiffness of the structure to reasonable extent.

Discrete damage probabilities obtained from Barbat, 2008 and HAZUS, 2003 have shown same trend for all three models considered for study.

7. CONCLUSIONS

Based on results and discussions, the following conclusions can be drawn.

1. Fundamental natural time period for partially infill RC frame is observed to be decreased in comparison with bare frame for first mode. Time period of building is reduced resulting into higher stiffness.
2. Base shears for partially infill RC frame are observed to be increase w.r.t. bare frame. While, displacements for RC frame with partially infill w.r.t. bare frame.
3. Damage probabilities of slight, moderate and extensive damage state of partially infill RC frame are decreased w.r.t. bare frame.
4. RC frame with infill arranged at periphery is found to be most efficient providing the least damage probability due to symmetrical arrangement of infill at outer extremities resulting into better resistance against twist during earthquake.
5. Hence, this proves that, the presence of non-structural masonry infill walls can modify the seismic behavior of R.C.C. framed building to large extent.
6. Comparison of fragility curves obtained from Barbat et al., 2008 and HAZUS, 2003 shown close association between each other. While HAZUS damage state give lower probability of failure for slight, moderate and extensive damage state and for complete damage state, curves are coincide with each other due to same complete damage state for both criteria.

7. Discrete damage probabilities obtained from Barbat, 2008 and HAZUS, 2003 have shown same trend for all models considered for study.

REFERENCES

[1] Ray Kai Leung Su and Chien-Liang Lee, "Development of seismic fragility curves for low-rise masonry infilled reinforced concrete buildings by a coefficient-based method", Journal of Earthquake Engineering and Engineering Vibrations, Vol.12, No.2, 2013, pp. 319–332.

[2] FEMA 2012, "Seismic performance assessment of buildings volume 1 – methodology", FEMA P-58-1, prepared by, Applied Technology Council, California 94065.

[3] Kursat Kinali and Bruce R. Ellingwood, "Seismic fragility assessment of steel frames for consequence-based engineering: a case study for Memphis, TN", Journal of Engineering Structures., Vol. 29, 2007, pp. 1115–1127.

[4] Astriana L., Sangadji S., Purwanto E., Kristiawan S.A., "Assessing Seismic Performance of Moment Reisting Frame and Frame-Shear wall system using Seismic Fragility Curve", Suistainble Civil Engineering Structures and Construction Materials, SCESCM-2016, Procedia Engineering 171, Elsevier, 2017, pp. 1069-1076.

[5] Barbat A.H., Pujaades L.G. and Lantada N., "Seismic Damage Evaluation in urban areas using Capacity Spectrum Method: Application to Barcelona", Soil Dynamics and Earthquake Engineering, 2008, pp. 851-865.

[6] HAZUS®MHMR4, Technical manual, "Multi-hazard Loss Estimation Methodology Earthquake Model", 2003, Federal Emergency Management Agency, Washington, D.C., United States.

[7] ATC 40, Volume 1 , "Seismic evaluation and retrofit of concrete buildings", Applied Technology Council, 1996, Seismic Safety Commission, State of California.

[8] Hemant B. Kaushik, Durgesh C. Rai and Sudhir K. Jain, "Rational Approach to Analytical Modeling of Masonry Infill in Reinforced Concrete Framed Buildings", The 14th World Conference on Earthquake Engineering October 12-17, 2008, Beijing, China.

[9] D. Das and C.V.R. Murthy, "Brick Masonry infills in seismic of RC framed buildings: Part 1- Cost implications", The Indian Concrete Journal, July 2004. Vol. 78, No. 7, pp. 39-43.

[10] R. Davis, P. Krishnan, D menon, A. M. prasad, "Effect Of Infill Stiffness on Seismic Performance of Multi-Storey RC Framed Buildings In India", 13th World Conference on Earthquake Engineering Vancouver, B.C., Canada, August 1-6, 2004, Paper No. 1198.

[11] Samoilă Diana M., "Analytical Modelling of Masonry Infills", Acta Technica Napocensis: Civil Engineering and Architecture, Vol. 55, No. 2, 2012, pp. 127-136.

[12] V.P. Jamnekar and P.V. Durge, "Seismic Evaluation of Brick Masonry infill," International Journal of Emerging Trends in Engineering and Technology (IJETET), 2013, Vol. 2, No. 1, pp. 1-6.

[13] FEMA 356-2000, , "Prestandard and Commentry for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency", Washington D.C.

[14] Mehmet Inel and Hayri Baytan Ozmen, "Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings", Journal of Engineering Structures, Vol. 281, 2006, pp. 494–1502.

[15] Prakash V., "Whither Performance-based Engineering in India?", ISET Journal of Earthquake Technology, 2004, Paper No. 447, Vol. 41, No. 1, pp. 201-212.

[16] IS: 1893(Part I)-2002, "Criteria for Earthquake Resistant Design of Structure, General Provisions and Buildings", Bureau of Indian Standard, New Dehli.

BIOGRAPHY

Ms. Punashri Prakash Phadnis presently working as an Assistant Professor in the Civil Engineering at Department of Technology, Shivaji University, Kolhapur, Maharashtra, India. She secured B.E. (Civil Engineering) in 2004 from Shivaji University, Kolhapur. She received M.Tech in Structural Engineering from Visveswaraiah Technological University, Belgaum in the year 2008 and pursuing Ph.D. in Seismic Analysis from Shivaji University, Kolhapur. Her interest includes Seismic Analysis of Structures, Analysis of Steel-Concrete Composite Structures, Bridge Analysis and Design.