© October 2025| IJIRT | Volume 12 Issue 5 | ISSN: 2349-6002

Seismic Performance and Vulnerability Assessment of Vertically Irregular Buildings

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Abstract: **In the present study a non-linear static method of analysis has been performed to estimate the behaviour of a building having vertical geometric irregularity. A comparison is made between a structure with masonry infill wall modelled, as against a structure without considering infill wall in it. This paper also focuses on studying the vulnerability of both structures and their responses to seismic parameters. It is observed that structures with infill wall panels perform better than bare frame structures, both in terms of seismic capacity, controlled displacement and better performance during various damage states. However, configuration of the building plays significance role under various damage state. As highlighted in the paper, the lesser step buildings perform poor during slight and moderate damage state, in comparison to higher stepped structures. On the contrary, they perform better under extensive and complete damage state.**

Keywords: **Seismic performance, vertical irregularity, seismic vulnerability, pushover analysis.**

1. INTRODUCTION

Seismic performance evaluation is based upon individual deformation capacity of an element as well as overall structural deformation capacity. Conventional seismic design in codes of practice is entirely force-based, with a final check on structural displacements. Seismic design follows the same procedure, except for the fact that inelastic deformations may be utilized to absorb certain levels of energy leading to reduction in the forces for which structures are designed. This leads to the creation of for over-strength, energy absorption and dissipation as well as structural capacity to redistribute forces from inelastic highly stressed regions to other less stressed locations in the structure and effects not explicitly considered in code applications. Although the code requires special ductile detailing, it does not provide a means to determine how the structure will perform under severe earthquake conditions. This paper highlights a procedure for evaluating the behavior of a structure and its response when subjected to seismic forces. Following terms are briefly defined to understand their implication on the seismic performance.

1.1 Response Reduction Factor (R):

The concept of response reduction factor (R) is also commonly known as force reduction factor, has emerged as a single most important number, reflecting the capability of the structure to dissipate energy through inelastic behavior. This factor is unique and different for different type of structures and materials used. Hence classification of response reduction factor for various structural systems is extremely important to do evaluation based on demand (earthquake ground motion) and capacity of the structure. Many design codes such as ASCE 7, Eurocode 8, IS 1893:2002 etc. do not consider non-linearity of structure fully in design procedures. Design methodologies used in these codes incorporate elastic forced based analysis. The factor R is an empirical factor intended to account for damping, over strength, and the ductility inherent in the structural system at displacements great enough to surpass initial yield and approach the ultimate load displacement of the structural system. Response reduction factor considers the nonlinearity of structure and reduces the elastic response of structure. As per global standards and codes such as ATC-40, FEMA 273 this factor has been defined as function of ductility factor, strength factor, redundancy factor and damping factor.

$$R = R_s \times R_\mu \times R_\xi \times R_R$$

where, Rs is strength factor, Rμ is ductility factor, Rξ is damping factor and RR is redundancy factor.

1.2 Performance Based Design:

In performance-based design, the limit states are typically known as structural 'performance levels', which in combination with seismic 'hazard levels' define the performance objective for a structure. The performance levels are defined based on the structure type and its intended functions. The purpose of Performance-Based Seismic Design (PBSD) is to give a realistic assessment of how a structure will perform when subjected to either particular or generalized earthquake ground motion. The various performance levels are IO (Immediate occupancy), LS (Life safety), CP (Collapse prevention). Using performance point global response of structure and individual component deformations compared to limits in light of the specific performance goal for a building can be assessed.

1.3 Vulnerability of Structure

The Seismic Vulnerability of a structure is described as its susceptibility to damage due to ground shaking of a given intensity. The aim of vulnerability assessment is to predict the seismic damage states. Figure 1.1 shows typical fragility curves which indicate various damage states in structure. Vulnerability curves are based on the nonlinear analysis of structural model under earthquake loading. HAZUS methodology of federal emergency management agency (FEMA, 2003) which is used to develop the fragility curves of the buildings. In which analytical capacity curves are used with assumed variability in demand, capacity and damage state thresholds.

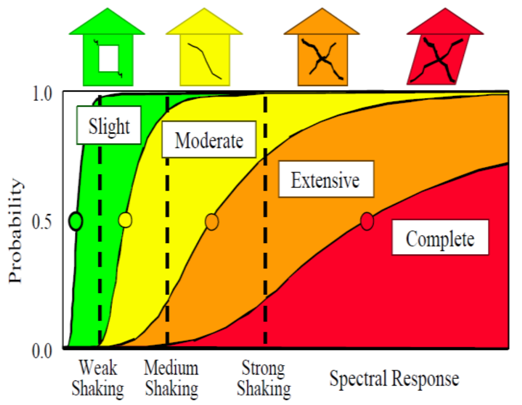


Fig 1: Fragility curve (HAZUS-MH-MR-01, FEMA, 2003)

2. METHODOLOGY

Following procedures are be followed to assess seismic parameters of the structure:

1. Seismic analysis and ductile detailing of all reinforced concrete frame structures by using IS1893:2002 and IS: 13920:1993.
2. Defining material nonlinearity, geometric nonlinearity and nonlinear loading to the building models.
3. Nonlinear analysis (Pushover analysis) using SAP2000 of all building models for nonlinear seismic performance assessment.
4. Calculation of seismic response reduction factor (R), over strength factor, ductility reduction factor from nonlinear analysis using pushover method.
5. Calculating seismic vulnerability assessment of models by using HAZUS fragility curve method from data obtained by nonlinear analysis.

2.1 Analysis

In this section, calculation of seismic coefficient, modelling of the reinforced concrete building with and without infill, calculations of response reduction factor and seismic damage vulnerability are presented. Seismic analysis of bare frame and unreinforced masonry infill frame structures are done by considering equivalent static method and response spectrum method of earthquake analysis as per IS1893:2016.

2.2 Seismic Coefficient, A_h: as per IS 1893 (Part 1): 2016

As per IS-1893:2016, clause 6.4.2 design horizontal acceleration spectrum value using fundamental natural period T_a in the considered direction of vibration is given as follows:

$$A_h = \frac{Z \times I}{2R} \times \left(\frac{S_a}{g}\right)$$

where;

Z = Zone factor, I = Importance factor, R= Response reduction factor, $\left(\frac{S_a}{g}\right)$= Average response acceleration coefficient depends upon fundamental natural period,

Fundamental natural time period (T_a) of equivalent static method for Moment resisting frame with unreinforced masonry brick infill panel,

$$T_a = \frac{0.09h}{\sqrt{d}}$$

Fundamental natural time period (T_a) of equivalent static method for moment resisting frame without infill panel,

$$T_a = 0.075 \times h^{0.75}$$

2.3 Diagonal Compression Strut for Masonry Infill Panel

The masonry infill walls are modelled as diagonal compression member strut with appropriate mechanical properties. As per IS 1893 (P1)-2016, the equivalent width of diagonal strut as shown in figure 7.1.is given as;

$$w_{ds}=0.175\ \alpha_h^{-0.4}\ L_{ds}$$

where, w_{ds} = Equivalent width of diagonal strut, α_h = Coefficient used to determine equivalent width of infill strut and can be obtained as;

$$\alpha_h = h\left(\sqrt[4]{\frac{Em\ t\ \sin 2\theta}{4\ Ef\ Ic\ h}}\right)$$

where, t =Thickness of masonry infill, h=Height of masonry infill, I_c =Moment of Inertia of adjoining Column, L_{ds}=Length of diagonal strut, r_{inf}= Diagonal length of masonry infill, h= clear height of URM infill between top beam and bottom floor slab, θ= angle of diagonal strut with the horizontal, E_f= Modulus of elasticity for RC MRF, E_m = Modulus of elasticity for masonry infill (in MPa) shall be taken as:

$$E_m = 550\, f_m$$

where f_m is the compressive strength of masonry prism (in MPa) obtained as per IS 1905 or given by the expression

$$f_m = 0.433\, f_b^{0.64} f_{mo}^{0.36}$$

f_b = compressive strength of brick (in MPa)

f_{mo} = compressive strength of mortar (in MPa)

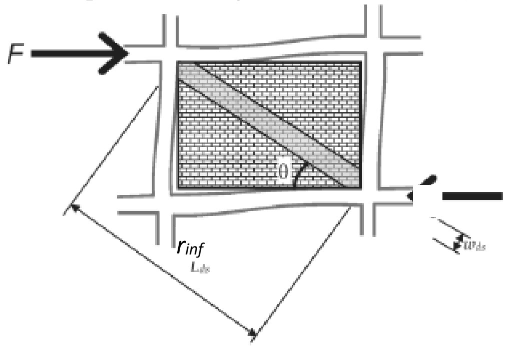


Fig 2: Equivalent diagonal strut of URM infill wall

2.4 Pushover Analysis

Performance assessment of the designed frames is carried out using nonlinear static pushover analysis. The modelling of the designed frames for nonlinear analysis is done in the Program SAP2000 Nonlinear. Pushover analysis is an approximate analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached. The roof displacement is plotted with base shear to get the global capacity curve (Fig 3).

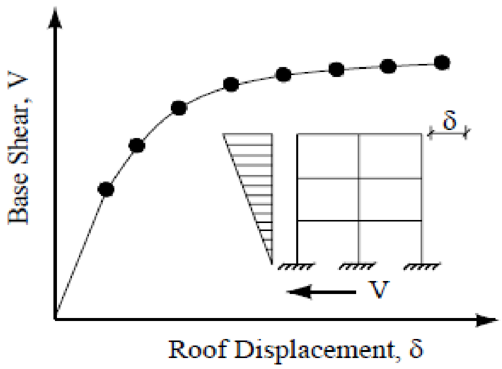


Fig 3: Pushover Curve of a Structure

2.5 Capacity Spectrum Method

This method compares the capacity of the structure (in the form of a pushover curve) with the demands on the structure (in the form of response spectra) as shown in figure 4. The graphical intersection of the two curves approximates the response of the structure, which is knowns as performance point of structure. The performance is dependent on the manner that the capacity is able to handle the demand. In other words, the structure must have the capacity to resist the demands of the earthquake such that the performance of the structure is compatible with the objectives of the design.

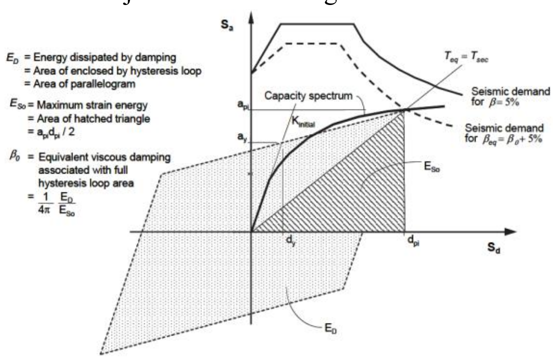


Fig 4: Graphical representation of Capacity Spectrum Method

2.6 Seismic Vulnerability

As per HAZUS, MH-MR-01-2003, fragility curves are lognormal distributions that represent the probability of being in or exceeding a given damage state, is given as

$$P[d_s/S_d] = \Phi\left[\frac{1}{\beta_{ds}}\right] \ln\left(\frac{S_d}{S_{d,ds}}\right)$$

where, *Sd* is spectral displacement, *Sd*, *ds* are the median spectral displacement for damage state ds, Φ is a normal cumulative distribution function, and βds is the standard deviation of the natural logarithm of the spectral displacement for damage state ds, which defines as follows:

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

where, β_C is the lognormal standard deviation parameter representing variability in the capacity properties of the building, β_D represents the variability in the demand spectrum due to spatial variability of the ground motion, and $\beta_{M(ds)}$ represents the uncertainty in the estimation of the damage state threshold.

To define damage state thresholds, HAZUS methodology have proposed a simpler approach as per table 1 based on yield and ultimate spectral displacement of the buildings. The yield spectral displacement (Sd_y) and ultimate spectral displacement (Sd_u) are obtained analytically from the bi-linearization of capacity curves.

Table-1: Damage State as per HAZUS

Damage state	Spectral displacement
Slight damage	$0.7Sd_y$
Moderate damage	Sd_y
Extensive damage	$Sd_y + 0.25(Sd_u - Sd_y)$
Complete damage	Sd_u

3. PROBLEM STATEMENT

Performance based nonlinear seismic analysis and vulnerability assessment of RC structures of six different models with and without masonry condition of various vertical steps as shown in figure 5.1 to 5.3 are considered in the present study.

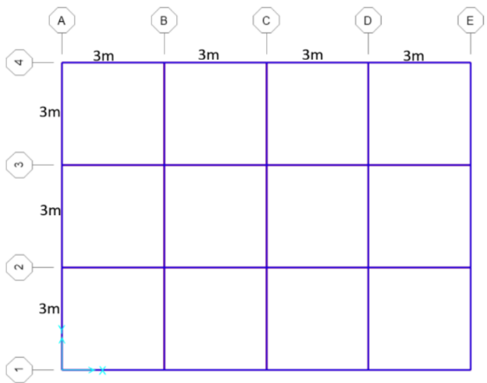
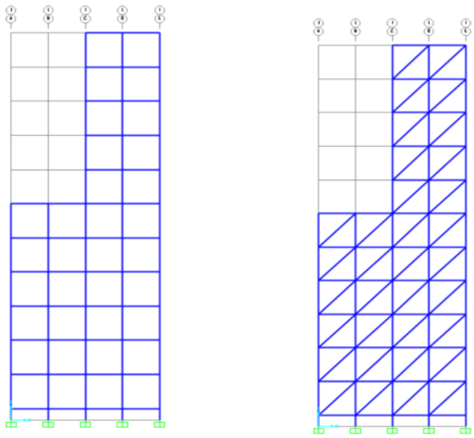
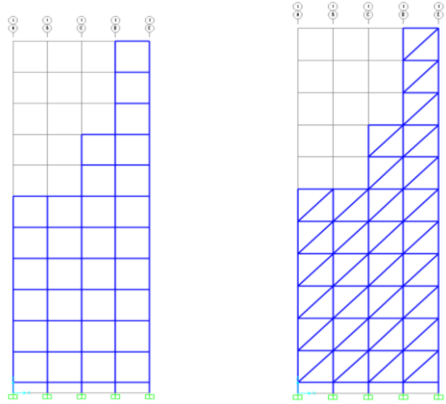


Fig 5.0: Plan of all buildings



BM-1A BM-1B

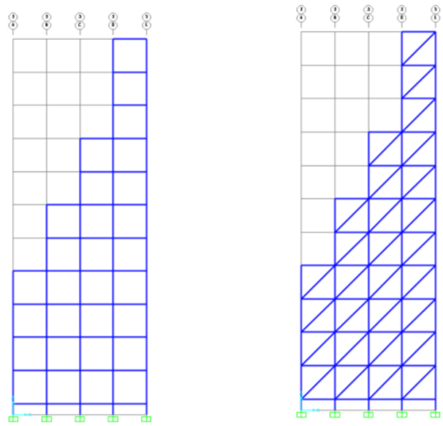
Fig 5.1: Elevation of buildings (BM-1A, BM-1B)



BM-2A BM-2B

Fig 5.2: Elevation of buildings (BM-2A, BM-2B)

Bare frame building model of G+10 storey with steps 1,2 and 3 are represented as BM-IA, BM-IIA and BM-IIIA respectively, similarly unreinforced masonry infill frame building model of G+10 stories with step 1, 2 and 3 are represented as BM-IB, BM-IIB and BM-IIIB respectively. Table 2 shows the details of various input parameters considered in present study.



BM-3A BM-3B

Fig 5.3: Elevation of buildings (BM-3A, BM-3B)

Table 2: Design input data

Description	Design data		
Building model	BM-IA and BM-IIA	BM-IB and BM-IIB	BM-IIIA and BM-IIIB
Floor to floor height (m)	3	3	3
Height of structure (m)	34	34	34
Plan dimensions (m x m)	12 × 9	12 × 9	12 × 9
No. of steps	1	2	3
Size of column (mm x mm)	300x600	300x600	300x600
Size of beam (mm x mm)	230x450	230x450	230x450
Slab thickness (mm)	150	150	150
Floor finish (kN/m^2)	1	1	1
Internal brick wall thickness (mm)	150	150	150
External brick wall thickness (mm)	230	230	230
Live load (kN/m^2)	2	2	2
Seismic zone	2	2	2
Zone factor (*Z*)	0.16	0.16	0.16
Response reduction factor (*R*)	5	5	5
Damping ratio (%)	5	5	5
Soil type	Medium	Medium	Medium

4. RESULTS AND DISCUSSION

Results of all six models considered in study are presented herewith, highlighting the impact of step and infill considered to various building parameters, viz. Time Period, Base Shear, Capacity Curve, Response Spectrum, Performance Point and vulnerability.

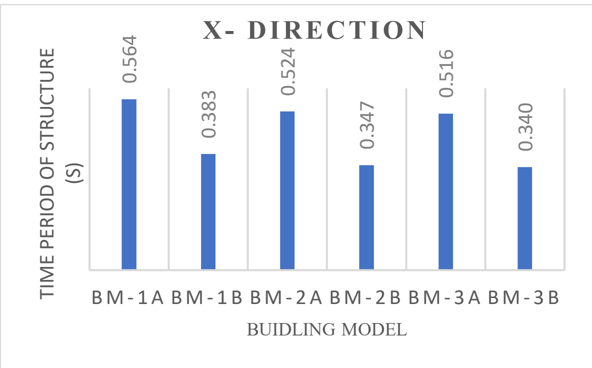


Fig 6: Time Period variation in all the building models along X-Direction

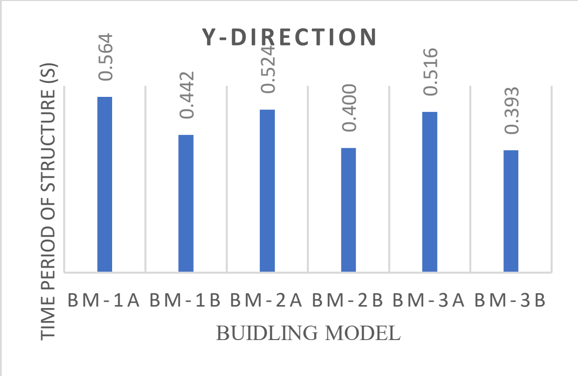


Fig 7: Time Period variation in all the building models along Y-Direction

It is observed that with increase in no of steps, Time Period reduces for both Bare frame structures as well as structures with infill panels. Also, Time period of infill panel structures in Y direction is more than Time period of the structure in X direction, even when the structure is stepped in X direction.

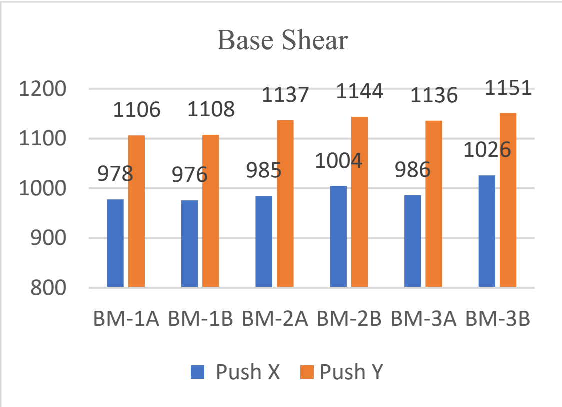


Fig 8: Base Shear

As highlighted in Fig.8, Base shear of the structure increases with increase in number of steps in both bare frame and infill wall structures.

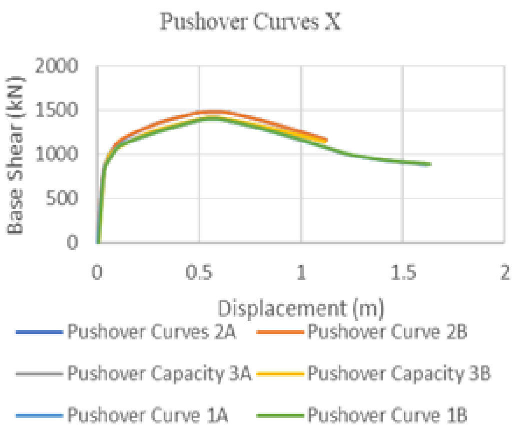


Fig. 9: Pushover Curves for X direction

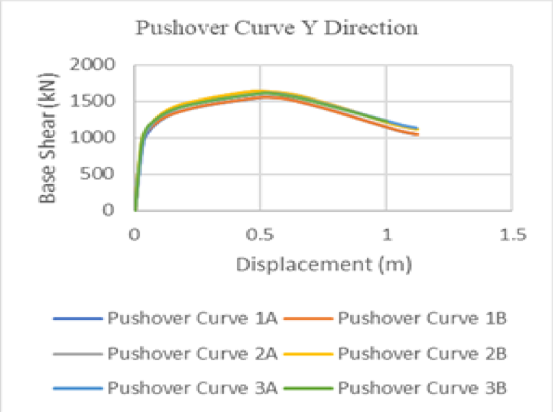


Fig 10: Pushover Curves for Y direction

All buildings show similar initial stiffness, indicating comparable structural rigidity in the early elastic range. BM-1B has the best ductility and energy dissipation capacity. BM-2A and BM-2B are stronger in terms of peak base shear. BM-1A and BM-1B have Less strength, but better deformability. BM-3A and BM-3B are Intermediate in both strength and ductility.

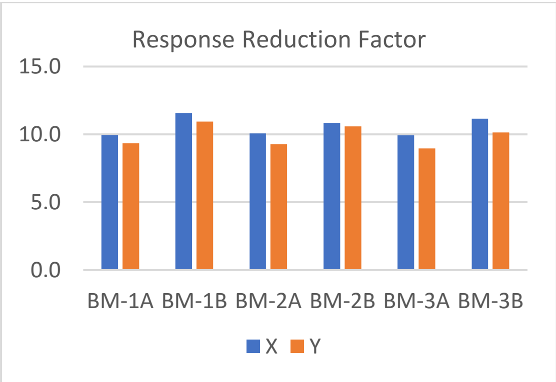


Fig 11: Response Reduction Factors

It is observed that R value increased for buildings with infill wall panels. Also, R value for bare frame obtained from dynamic analysis is almost double than code recommended value of 5.

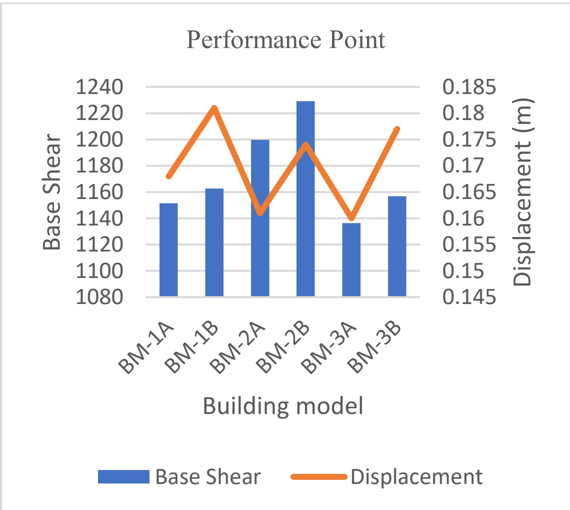


Fig 12: Performance Point

It is observed that BM-1B, BM-2B, BM-3B show higher base shear capacity than their corresponding bare frame models. Displacement is slightly higher in BM-1B and BM-3B compared to their bare frame counterparts. However, BM-2B (which also has the highest base shear) shows moderate displacement, indicating a good balance of strength and flexibility.

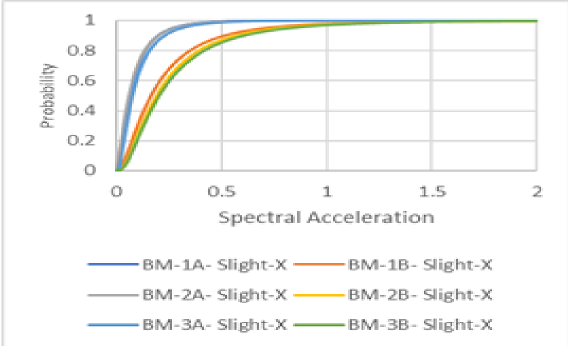


Fig 13: Graph of Damage level Slight X

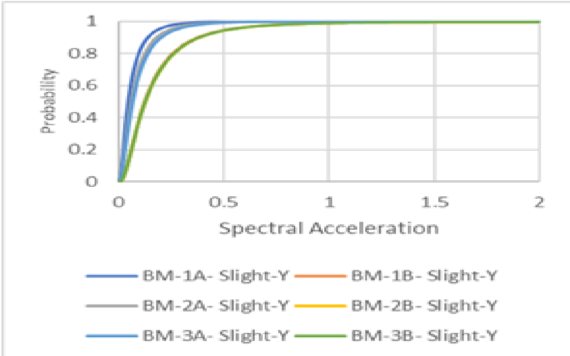


Fig 14: Graph of Damage level Slight Y

BM-1A and BM-2A have steep and left-shifted curves, indicating that even at lower demand, the probability of slight damage is high. BM-3A performs slightly better among bare frames, as its curve is less steep and slightly right-shifted. BM-1B, BM-2B, BM-3B show more gradual slope and are right-shifted compared to bare frames. BM-3B shows the best performance, indicating it is least likely to experience slight damage under lower seismic demands.

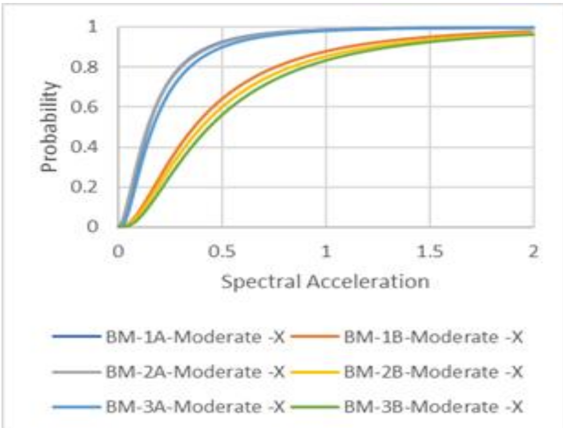


Fig 15: Graph of Damage level Moderate X

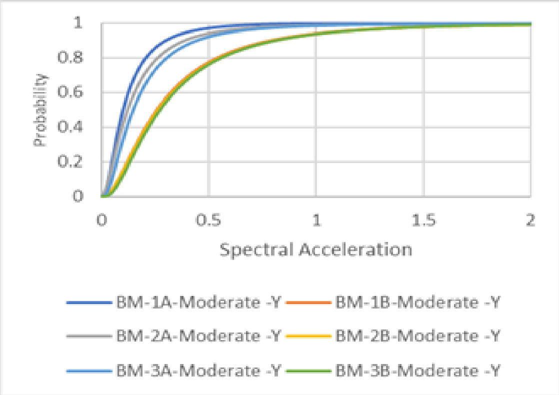


Fig 16: Graph of Damage level Moderate Y

BM-1A, BM-2A, BM-3A curves rise faster at lower intensity values, meaning they reach moderate damage earlier compared to infilled buildings. BM-1B, BM-2B, BM-3B curves shift to the right (slower rise). This indicates better resistance to moderate damage at the same intensity level due to the contribution of infill walls providing additional stiffness and strength. BM-1A are more vulnerable, showing earlier probability of moderate damage. BM-3A show slightly improved resistance. BM-1B still performs better than BM-1A but is more vulnerable than BM-2B and BM-3B. BM-3B shifts furthest right, showing the best resistance.

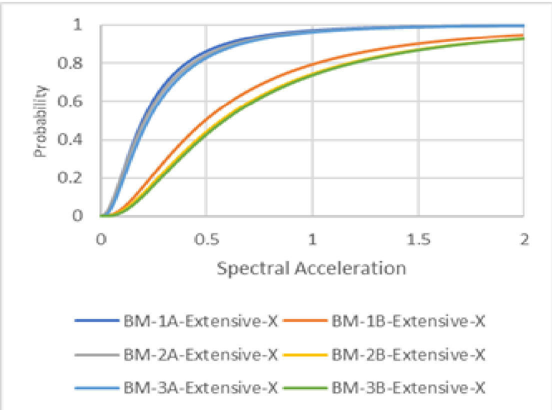


Fig 17: Graph of Damage level Extensive X

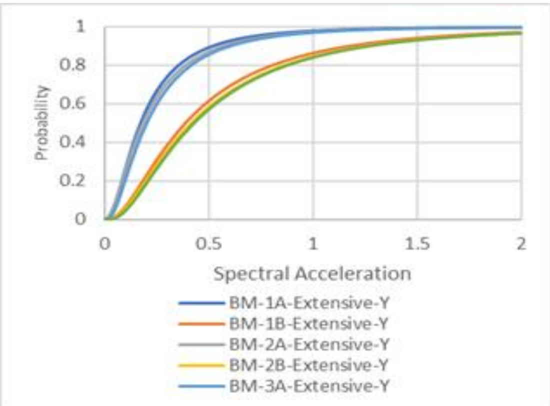


Fig 18: Graph of Damage level Extensive Y

BM-1A, BM-2A, BM-3A curves rise faster (shifted upward compared to infill panelled structures). This means bare frames are more vulnerable and reach higher damage probabilities at lower intensity levels. BM-1B, BM-2B, BM-3B curves rise more gradually. They exhibit improved resistance, delaying the onset of extensive damage compared to bare frames. BM-3A shows the highest probability of reaching extensive damage earliest, reflecting its higher vulnerability. BM-3B performs significantly better than BM-3A, though still not as strong as shorter infilled buildings.BM-1B, BM-2B show the best resistance overall.

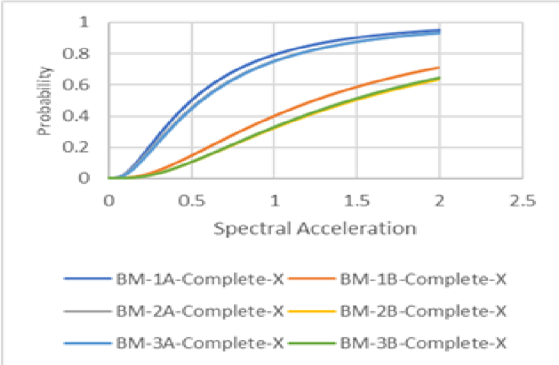


Fig 19: Graph of Damage level Complete X

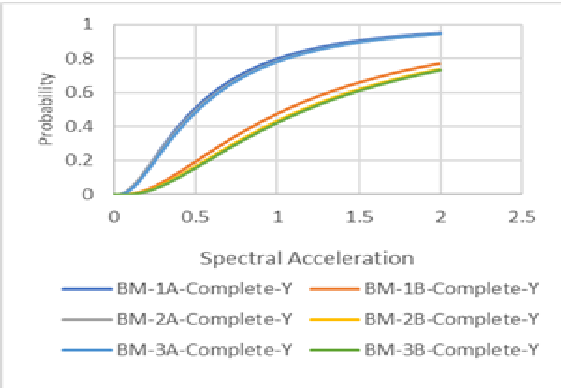


Fig 20: Graph of Damage level Complete Y

BM-1A and BM-3A show the steepest and highest vulnerability curves, reaching 90% probability of complete damage by spectral acceleration of 2.0. BM-2A is less vulnerable compared to BM-1A and BM-3A, but still more vulnerable than BM-2B. Among the infilled structures the probability of complete damage remains consistently lower, peaking around 60%–70% at intensity 2.0, indicating significant benefit from infill walls. BM-3A and BM-3B show higher vulnerability than BM-1 and BM-2 at comparable intensities. This indicates that with increase in steps structures are more vulnerable, even when infilled, although infill still improves their resistance.

4. CONCLUSION

The studies indicate that infilled structures perform better than bare frame structures. No of steps (in elevation) also affects the behaviour of the structure. The overall results are summarised as under:

1. Infill wall panels significantly improve the lateral load resistance of buildings, as evidenced by the higher base shear values in BM-1B, BM-2B, and BM-3B compared to their bare frame versions.
2. While displacement increases slightly in infilled structures (BM-1B, BM-3B), this trade-off is minor when compared to the substantial gain in base shear capacity.
3. Bare frame structures (BM-1A, BM-2A, BM-3A) are more vulnerable, with damage likely to occur at much lower seismic intensities. Among all, BM-3B consistently shows the best performance, followed by BM-2B for slight damage state.
4. BM-3A, BM-3B show slightly improved resistance compared to BM-1A, BM-1B, for moderate damage state.
5. BM-3A, BM-3B are more vulnerable than BM-1A, BM-1B, indicating that increase in steps increases risk of damage, for extensive damage state.
6. No. of steps also influences vulnerability-more flexible frames (BM-3A, BM-3B) are more prone to complete damage than shorter ones, regardless of infill.
7. BM-2B shows the best overall performance, combining highest base shear with moderate displacement, making it the most seismically efficient structure among the six

REFERENCE

[1] Applied Technological Council (ATC – 40) (1996), "Seismic Evaluation and Retrofit of concrete buildings," Applied Technology Council, Redwood City, USA

[2] Kappos, J. (1998), "Evaluation of behaviour factors based on ductility and over strength studies," Engineering Structures, vol 21, no 9, pp. 823–835.

[3] Chaulagain, H., Enrico, S., H., Rodrigues H., and Varum, H. (2014), "Response Reduction Factor of Irregular RC Buildings in Kathmandu Valley," Earthquake Engineering and Engineering Vibration, vol 13, no 3, pp. 455-470.

[4] Das, D., and Murthy, C. V. R. (2004), "Brick masonry infills in seismic design of RC frame buildings," The Indian Concrete Journal, vol 77, no 5, pp. 31-38

[5] Erberik, M. (2008), "Fragility-based assessment of typical mid-rise and low-rise RC buildings in Turkey," J. Engineering Structures, vol 30, no 12, pp 1360–1374.

[6] Girgin, K., and Darılmaz, K. (2007), "Seismic response of infilled framed buildings using pushover analysis," ARI the bulletin of the Istanbul technical university, vol 54, no 5, pp.105-112