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Seismic Performance of RC Buildings with Different Positions of Lift Core Wall and Added Shear Walls

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Abstract

Selection of structural systems that will be capable of resisting the earthquakes is important to prevent the structural cracks in minor shaking and prevent collapse in major shaking to create safety in life. Properly located and well-designed structures with shear walls as lateral load resisting members increase the lateral stiffness in plane of wall which would optimize the columns and would have greater living spaces in the buildings. Use of these walls in casing of elevators has been in great practice, since it affects the architectural design of the building. Easy access of lifts increases the functionality of buildings. Lift core placed in best position in the building, will greatly influence the lateral stiffness of building so that the vulnerability of buildings will be less compared to bare frames. Eccentric positioning of lift core will create eccentricity in the building, which would require the greater reinforcements. With increase in eccentricity the building may bear the torsional irregularities. So, there is need of extra shear walls to balance the irregularities. Bidirectional earthquake excitation analysis is necessary for these types of buildings. IS 1893:2016 is used in design. For various damage states drift limits are taken from FEMA 356 2000 and median values of displacements are taken as per HAZUS 4.2 SP3. This research work presents the vulnerability due to eccentric positioning of lift core in symmetrical reinforced concrete frame. The torsional irregularities are needed to be removed with optimum positioning of extra shear walls. The reduction in vulnerability of buildings due to added walls is studied. The probability of exceeding the collapse damage states at 0.4g PGA are greater for FEMA 356 2000 than HAZUS 4.2 SP3. The percentage increase in vulnerability for lift core at corner is 112.51% as compared to its centric position.

Keywords

Reinforced concrete, Shear wall, Lift core, Torsional irregularity, Bidirectional earthquake, Vulnerability.

1. Introduction

A shear wall with RC frame will encounter the effects of lateral loads acting on a structure due to earthquake, wind etc. The size of the columns gets reduced considerably and can be changed to a large extent at different floors with the use of shear wall in frame [1]. Lateral forces are decreased when shear walls are put at the proper positions to frames [2]. With the addition of a shear wall, base shear increases and lateral displacement decreases [3]. Proper positioning of shear walls increase the strength and stiffness of a structure and can significantly impact the seismic behavior of frame structures [4]. Square shaped shear wall is the most effective with comparison to channel shaped, T shaped and I Shaped [5]. Constructing building with shear wall in short span at corner is economical [6]. L type shear wall is best in comparison with cross type shear wall and shear wall at periphery for G+5 symmetrical building with plan 16m*16m [7]. For rectangular sections, the fiber method predicts the nonlinear behavior of the structure at acceptable level [8]. Since the fiber model can replicate the development of plastification within the plastic hinge region, it is more accurate for simulating hysteretic behavior using fibre model than the assumption of a single element with concentrated hinges [9]. Location of shear wall at the edge of building resulted in heavy axial loads in columns with increase in drift and displacement of building [10]. Better seismic evaluation will be possible with the combination of nonlinear time history analysis and probabilistic assessment.Seismic performance of building and vulnerability assessment has been the interest with the increase of the computational efficiency. Staircases and elevators are to be provided in buildings at such location such that they more easily accessed. But the position of core wall may increase the eccentricity in the buildings giving the more torsional effects. So, extra shear walls are to be located in building to decrease the eccentricity and hence control the torsion. The torsional irregularities due to shifting of the lift core wall are to be analyzed and the analysis of building after balancing the torsion is also an important work to be done.

2. Methodology

During initial phase of study some mid rise buildings were surveyed in Kathmandu city. For initial assumption of slab thickness, shear wall thickness, beam and column dimensions those surveys helped .A symmetrical RC frame has no eccentricity in both direction in plan, so for this reason a symmetrical RC frame is selected. Number of bays were taken similar to those which were taken in similar type of works done in past. Since this study deals with variation in seismic performance due to change in lift core wall positions, door opening in lift core hasn't been considered and staircases are not modeled, which are the limitations of the study. Dual system of G+6 Symmetrical moment resisting frame of 5*5 bays with equal bay lengths of 4.5 m in both direction with constant storey height of 3m, with beam sizes 14"*18" and columns sizes 16"*16" and lift core wall of size 3m * 2.5m in X and Y direction at centre position in plan (11.25,11.25) from base to top with thickness of 250mm is selected as a base model (Model 1) as shown in figure 1. The grade of concrete is M25, and that of steel is Fe500, thickness of slab is 125mm. The seismic zone considered is V, response reduction factor is 5, soil type medium, and importance factor is 1. The modal damping is at 5%. The lift core wall is shifted in three different positions. Model 2, Model 3 and Model 4 have lift core's centre at position (11.25,21.375), (15.75,21.375) and (20.25, 21.375) respectively. Beams and columns are modeled as frame elements, slabs as shell elements and shear walls as wall elements in ETABS V18.1. For nonlinear modeling, plastic hinges are assigned to beams and columns as per ASCE 41-17 [11] at 0.45m from the ends and fiber hinges are assigned in wall sections. These models have been designed for the design combinations as per IS 456:2000 [12] and IS 1893 (part1): 2016 [13], used in this study as these buildings codes have been widely used in research and field works in Nepal from a very beginning. Moreover IS 1893:2016 has two criteria for a building to have torsional irregularities. The torsional irregularities in the building models are balanced from the approach of torsional sensitivity [14], that is higher modal mass participation due to rotation in first two fundamental modes of vibration are removed with addition of extra shear walls at optimum positions.

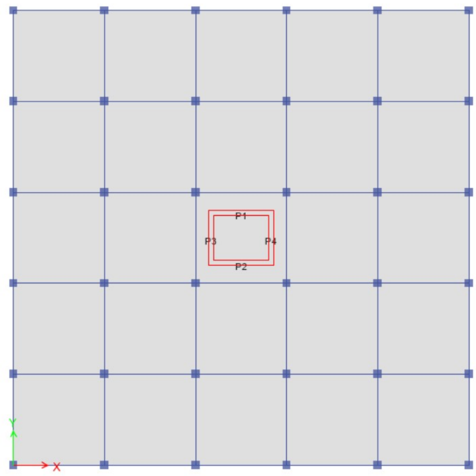

Figure 1: Model 1 (lift at centre)

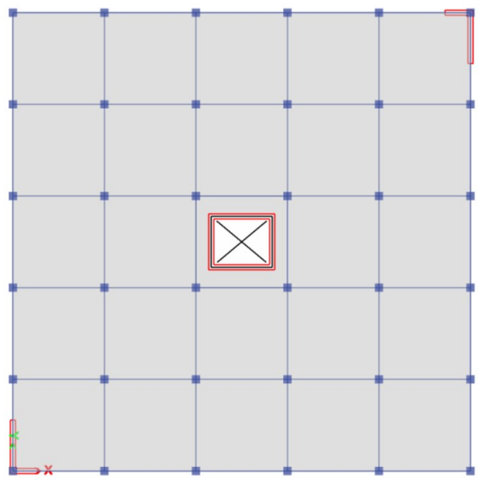

Figure 2: Model 5

Model 5, model 6, model 7 and model 8 are the models after balancing the torsional irregularities of models 1, models 2, models 3 and model 4 respectively by adding extra shear walls of 250mm thickness at suitable locations such that eccentricities will be less than 5% and the modal mass participation in first two fundamental modes are purely translations. In Model 5 the length of extra added walls are 2.5 m in X direction and 5 m in Y direction as shown in figure 2. In model 6 length shear wall is added is 4.5m at bay 3, at centre of edge opposite to liftcore. In model 7 the added wall are at corner(0,0), 3.635m in X direction and 2.0625m in Y direction. In Model 8 the added walls are at corner (0,0), 3.75m in X direction and 3 m in Y direction. Site specific seven earthquakes are selected from PEER databases and matched to the target response spectrum from IS 1893 (part1):2016 in Seismo match version 2021. Bidirectional earthquake load cases for fast nonlinear analysis, used because of its faster computation, for various levels of 0.2, 0.4, 0.6, 0.8, and 1 PGAs so that logarithmic interpolation can be done at 0.05 intervals of PGAs, has been used. The median values of displacements and beta values for midrise building category of concrete shear wall building are taken from HAZUS 4.2 SP3 [15] are in table 1. The drift limits from FEMA 356 2000 for immediate occupancy(IO), Life Safety(LS), and collapse prevention(CP) are 0.5%, 1% and 2% respectively. The fragility curves are plotted for both cases. The peak ground acceleration (PGA) for Kathmandu city is 0.4g [16, 17], so comparisons of vulnerability through fragility curves are done at 0.4g PGA value.

Table 1: Median Values of Displacements and beta

Damage states	Displacements(mm)	Beta
Slight	30.48	0.821
Moderate	64.262	0.77
Extensive	176.53	0.73
Collapse	457.2	0.91

3. Results

3.1 Eccentricity

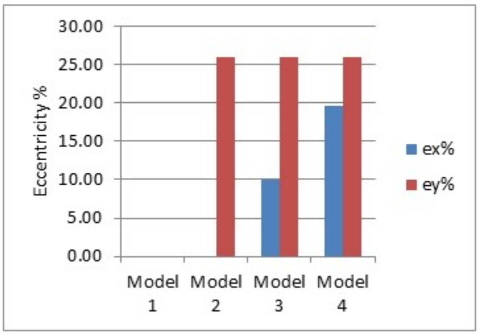

Figure 3: Eccentricity results

The eccentricity is increased with shifting the lift core away from the centre of building. Model 1 has zero eccentricity. Model 4 has greatest eccentricities in both directions. Models 6, 7 and 8 are such that the eccentricity values are less than 5% in both directions.

3.2 Modal mass participation ratios

Table 2: Modal mass participation for Model 1

Mode	Ux	Uy	Rz
1	0.0000	0.0000	0.8370
2	0.0000	0.7124	0.0000
3	0.7060	0.0000	0.0000
Sum	0.7060	0.7124	0.8370

Table 3: Modal mass participation for Model 2

Mode	Ux	Uy	Rz
1	0.4900	0.0000	0.3332
2	0.0000	0.7084	0.0000
3	0.2400	0.0000	0.4576
Sum	0.7300	0.7084	0.7908

Table 4: Modal mass participation for Model 3

Mode	Ux	Uy	Rz
1	0.4250	0.0990	0.3010
2	0.0998	0.6017	0.0068
3	0.2027	0.0122	0.4768
Sum	0.7275	0.7135	0.7846

Table 5: Modal mass participation for Model 4

Mode	Ux	Uy	Rz
1	0.3086	0.2781	0.2388
2	0.2851	0.4120	0.0100
3	0.1270	0.0000	0.5190
Sum	0.7210	0.7243	0.7680

Modal 1 has torsional irregularity as the fundamental first mode of vibration is dominated by the torsion. The other two modes of vibration in Model 1 are translational. In models 2 and 3 we can see that in the first mode of vibration, translation in X direction is coupled with rotation however the first modes are dominated by the translation in X direction. In modal 4, first mode of vibration has coupled translation in both direction and torsion. The translations in both directions are coupled in 2nd mode of vibration but there is no rotational participation.

Table 6: Modal mass participation for Model 5

Mode	Ux	Uy	Rz
1	0.6247	0.0778	0.0000
2	0.0770	0.6126	0.0000
3	0.0000	0.0000	0.7250
Sum	0.7017	0.6994	0.7250

Table 7: Modal mass participation for Model 6

Mode	Ux	Uy	Rz
1	0.0000	0.7053	0.0000
2	0.6600	0.0000	0.0364
3	0.0400	0.0000	0.6666
Sum	0.7000	0.7053	0.7030

Table 8: Modal mass participation for Model 7

Mode	Ux	Uy	Rz
1	0.0202	0.6805	0.0004
2	0.6733	0.0203	0.0023
3	0.0021	0.0002	0.6974
Sum	0.6956	.7010	0.7001

Table 9: Modal mass participation for Model 8

Mode	Ux	Uy	Rz
1	0.0761	0.6240	0.0006
2	0.6192	0.0757	0.0021
3	0.0018	0.0022	0.6952
Sum	0.6971	0.7019	0.6979

After the shear walls have been added in the initial models of buildings with lift core wall only, the mode participation mass ratio has been changed such that the first two modes of vibration are translational and the coupled translation and coupling of translation with rotation has been removed from the first two modes. All models the sum of modal mass participation for the first three modes has exceeded 65% in both X and Y direction, this means the models don't have irregular modes of oscillations. Since during ground motion earthquake excitation the mode participation has important role as the building oscillates in the fundamental modes, so after placing the shear wall at optimum location in the building with the core lift wall all the models have translational modes, which signify that these building are not subjected to torsion.

3.3 Inter storey drift ratios

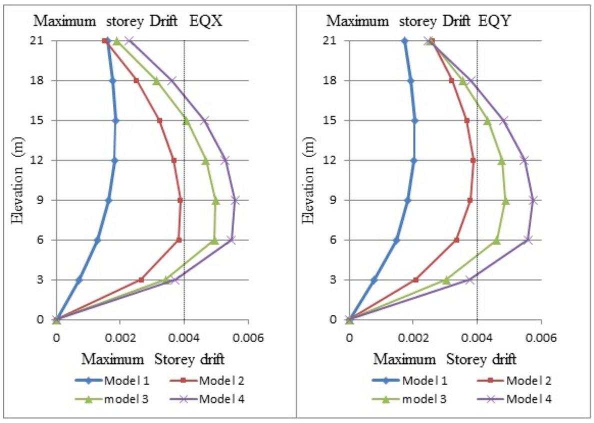

Figure 4: Interstorey Drifts

The maximum interstorey drift limit according to IS 1893:2016 is 0.004 times the storey height. Here we have obtained the values of maximum interstory drift ratio so it is to be compared with 0.004. Shifting the liftcore away from the centre increased the drift ratio out of which the model 3 and model 4 have crossed the limit value.

3.4 Dmax/Dmin Ratios

Table 10 presents the maximum displacement of top storey at one end and minimum displacement at the far end both in X direction for the linear static load cases EQX with eccentricity considered and the ratio is calculated to find if the building models suffer torsional irregularity.

Table 10: Torsional irregularity with Dmax/Dmin ratio

Model	Dmax (mm)	Dmin (mm)	Ratio	Remarks
Model 1	32.107	22.534	1.425	Regular
Model 2	63.806	17.141	3.722	Irregular
Model 3	81.273	12.308	6.603	Irregular
Model 4	91.470	9.499	9.629	Irregular
Model 5	27.956	22.978	1.217	Regular
Model 6	21.059	15.119	1.393	Regular
Model 7	21.256	16.885	1.259	Regular
Model 8	20.376	18.499	1.101	Regular

Torsional irregularities exist in model 2, model 3, model 4 since the ratio of maximum displacement at top storey in X direction to minimum storey at far end in same direction due to eccentricity considered EQX load case are greater than 1.5. Though model 1 had torsional irregularity due to rotational first mode of vibration, it has the ratio defined in this section less than 1.5. the building having the ratio greater than 1.5 would have higher response in the transverse direction due to loading at orthogonal direction because they are suffering from torsion in more extent. After placing the extra shear wall in these models the torsional irregularity has been removed as we can see the ratio for model5, model 6, model 7 and model 8 are less than 1.5. these models will have lesser torsional effect.

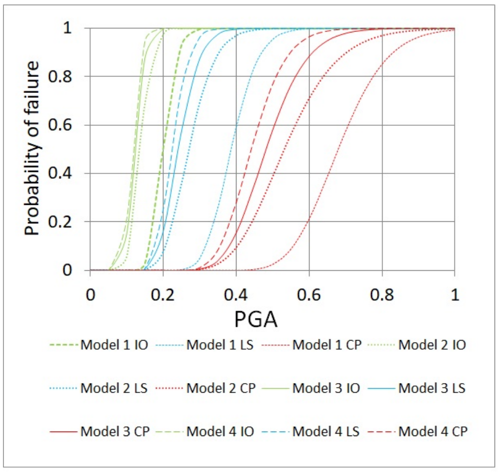

Figure 6: Fragility curves as per FEMA 356 2000

3.5 Fragility analysis

The incremental dynamic analysis curves showing the PGAs values in X axis and the drifts in Y axis is shown as figure 5 for Model 1.

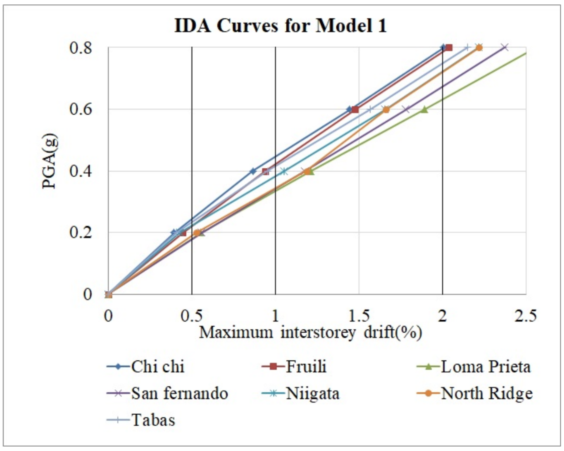

Figure 5: IDA curves for Model 1

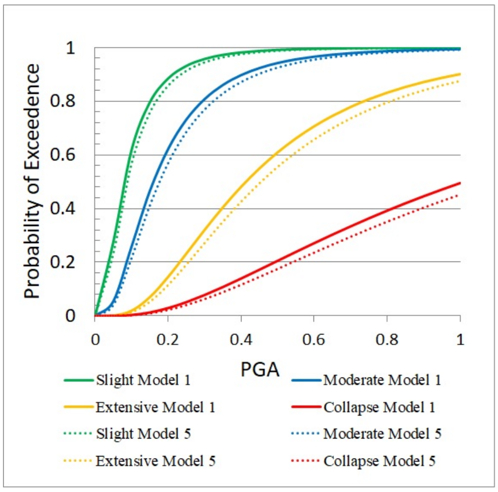

Figure 7: Fragility curves for Model 1 and Model 5 as per Hazus 4.2 Sp3

Mean and standard deviations of logarithm of PGAs at damage states for model 1 are shown in table 11.

Table 11: Mean and Standard deviation for Model 1

Damage States	Mean	Standard deviation
IO	-1.611	0.152
LS	-0.952	0.153
CP	-0.386	0.156

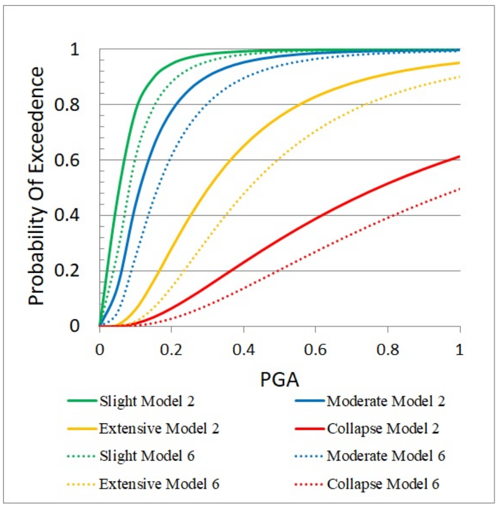

Figure 8: Fragility curves for Model 2 and Model 6 as per Hazus 4.2 Sp3

IDA curves were obtained for other models and the mean and standard deviations of logarithm of PGAs at the damage states were calculated. The average IDA curve when combined with mean and standard deviation give the fragility curves as shown in figure 6.

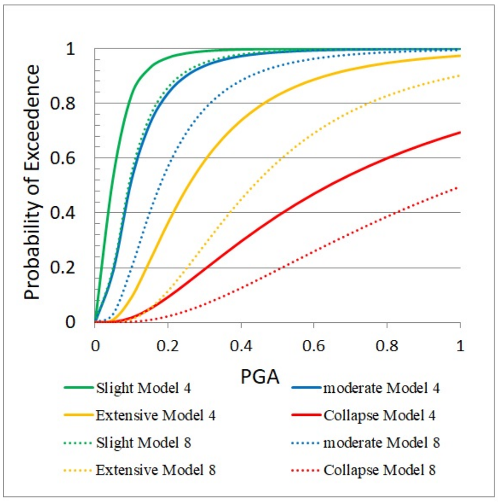

Figure 9: Fragility curves for Model 3 and Model 7 as per Hazus 4.2 Sp3

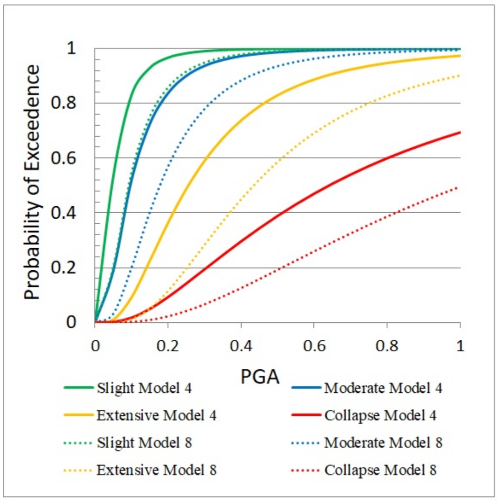

Figure 10: Fragility curves for Model 4 and Model 8 as per Hazus 4.2 Sp3

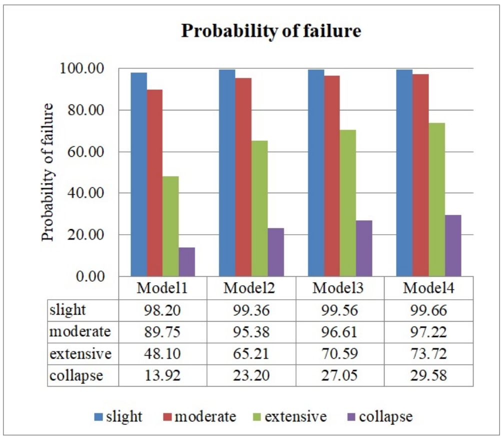

Figure 11: Probabilities of failure at 0.4g PGA as per HAZUS 4.2 SP3

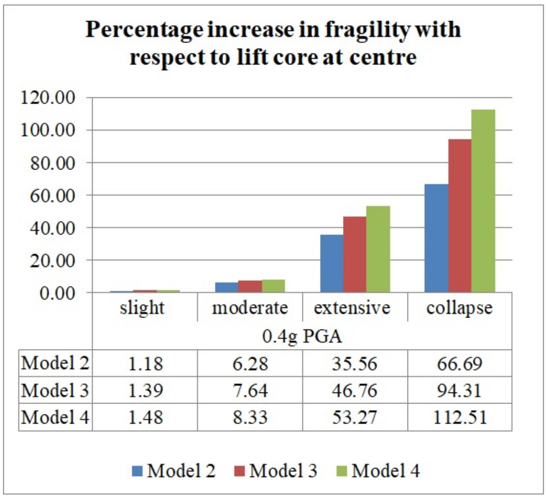

Figure 12: Percentage increase in fragility as compared to lift core at centre

Shifting the lift core wall away from the centre of building has less influence for slight damage limit state of HAZUS 4.2 SP3 which can be seen in the bar diagram in figure 11 as model 1, model 2, model 3, and model 4 have almost same probability of failure at 0.4g PGA. For the moderate damage state shifting the lift wall away from the centre in edge of building has increased the probability of failure slightly. Model 4 have the greatest eccentricity for both directions, so the displacement considered in one direction has been greatly influenced by the transverse directional displacement of the orthogonal excitation. For HAZUS's extensive damage state, the probability of failure has been increased by 35.56%, 46.76% and 53.27% for model 2, model 3 and model 4 respectively with respect to model 1. For the collapse condition, the probability of failure are increased by 66.69%, 94.31%, and 112.51% for model 2, model 3 and model 4 respectively with respect to model with lift core at center of the building.

The fall in vulnerability after balancing the torsional irregularities can be seen in fragility curves from HAZUS 4.2 SP3, which are shown by dotted lines in figure 7 to figure 10. At 0.4g PGA the fall in vulnerability in terms of probability of exceeding the damage states from HAZUS 4.2 SP3 after addition of extra shear walls are 0.64%, 2.87%, 11.54% and 16.83% for model 1, 1.18%, 5.9%, 26.45% and 40.28% for model 2, 1.67%, 8.43%, 35.95% and 53.16% for model 3 and 1.80%, 9.16%, 39.08% and 57.61% for model 4 for slight, moderate extensive and collapse damage states respectively. Increased Dmax/Dmin ratio required greater length of shear wall to balance the torsion in the building, so the added walls have increased the lateral stiffness and hence the reduction in vulnerabilities are greatest for Model 4.

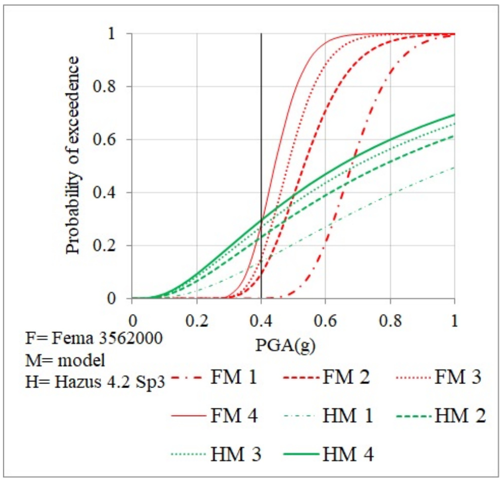

Figure 13: Fragility curves at collapse damage states for models(1-4)

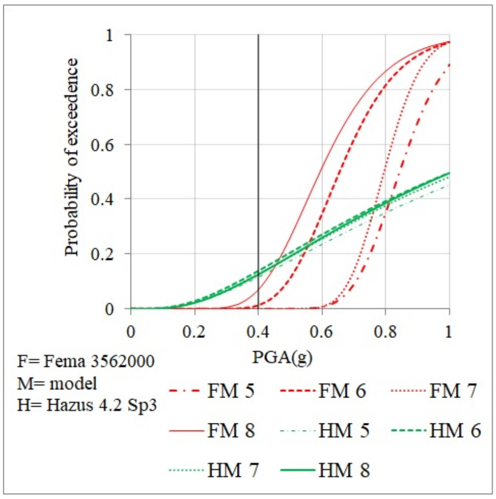

Figure 14: Fragility curves at collapse damage states for models(5-8)

The above fragility curves in figure 13 and figure 14 depict that the probability of exceeding the drift limits at collapse damage states are lesser for FEMA 356 2000 than the displacement limits from HAZUS 4.2 SP3 for 0.4g PGA. For the average of all the fragility curves plotted, the probability of collapse at FEMA damage state is 67.78% lower than that at HAZUS for 0.4g PGA. At lower PGAs value less than 0.3g the probability of exceedance of collapse prevention damage states are very small nearly zero for FEMA's drift limit of 2% .The fragility curves for drift limits have the probability of exceedance sharply increasing with increase in PGA while that for displacement are increasing uniformly. Slope of fragility curves from HAZUS 4.2 SP3 are lower than that for FEMA 356 2000 drift limits. At higher values of peak ground acceleration the probabilities of exceeding drift limits are higher.

4. Conclusions

Shifting the lift core away from the center increases the lateral displacements, interstorey drifts and creates torsional irregularities in building. Model 3 and Model 4 have interstorey drifts greater than 0.004. Probabilities of failures at 0.4g PGA for collapse damage states are greater for HAZUS 4.2 SP3 than FEMA 356 2000. Providing the extra shear wall for balancing the torsional irregularity makes the first two fundamental modes of translation and hence decreases the vulnerability.

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