SEISMIC PERFORMANCE OF RC DUAL SYSTEM BUILDINGS DESIGNED FOR INDIAN CODE

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Abstract

Reinforced concrete (RC) shear walls are frequently used in mid- and high-rise buildings to provide lateral resistance against earthquake demands. In India, as in other parts of the world, shear walls are often used in combination with moment resisting frames to control the inter-storey (IS) drift drift. Such buildings are commonly referred to as dual system buildings. The current Indian codes provide some guidelines for the design of dual systems. These guidelines are mostly restricted to the capacity-design verifications and to define minimum forces for design of moment-resisting frames, but no guidance is provided for proportioning and capacity design verification of shear walls. Further, the Indian seismic design code, like most other national codes worldwide, is based on force-based design philosophy and hence does not provide any framework to estimate the intended/expected seismic performance of such buildings. The objective of the present study is to determine the seismic performance and collapse fragility of RC dual system buildings designed with the Indian codes. The non-linear behaviour of shear walls, and columns of the moment-resisting frames, are modelled using fibre-hinge model, duly calibrated with experimental results available in the literature. The non-linearity in beams is modelled using an experimentally calibrated lumped plasticity model. Bi-directional incremental dynamic analyses (BIDA) are carried out to assess the collapse capacity of the buildings for a set of far-field ground motion records. The capacity curves obtained using BIDA are used to estimate the seismic fragility and collapse probabilities of the buildings. The wall area in these building models is varied to study the influence on their performance and on the collapse capacity. The performance of the buildings is considered to estimate the optimum area of shear walls as a fraction of the plan area. The results indicate that a minimum of 0.9 - 1% of the shear wall area in proportion to plan area should be provided in each direction of the building to achieve acceptable performance of the building in terms of probability of collapse and collapse resistance of the building at MCE (return period 2475 years) demand level.

Keywords: Collapse capacity; Design code; Dual systems; Fragility analysis; Shear walls

1. Introduction

RC shear walls with moment frames are commonly used as the primary lateral force-resisting system (LFRS) for mid and high-rise buildings in regions with moderate to strong earthquake ground shaking intensity. The moment frame in these buildings resist the lateral load involving beams and columns and their connections, while shear wall resists lateral loads in its in-plane direction. These walls provide substantial stiffness and strength as well as sufficient deformation capacity to the building to resist lateral motions. The contribution of shear walls to the lateral load resistance depends upon the relative stiffness and strength between shear walls and frames. Whenever the combined action of frames and walls is used to resist lateral load in RC buildings in a balanced fraction, the system is referred as "*Dual System*". The design of dual systems, apart from the design of the frames and shear walls, requires special consideration of the frame-shear wall interaction, since both tend to have different deformed shapes. While both frame and shear wall, share the storey shear forces in the lower storeys, they tend to oppose each other at higher storey levels [1]. These shear walls may be of any shape like planar shear walls (i.e., rectangular) and non-planar (i.e., L shaped, wall core or channel). Planar walls are usually used for dual system buildings with heights less than 73 m and are commonly designed using a linear-elastic approach [2].

There are several post-earthquake damage studies in the literature reporting the better performance of buildings with shear walls than buildings without structural walls [3 - 7]. These studies have concluded that the performance of these buildings can be improved if an adequate number of shear walls are provided in the two main directions of the buildings. Thus, shear wall area ratio, defined in terms of percentage of the area of the shear wall (in one direction) in proportion to the floor plan area, has been used as a critical parameter to measure the seismic performance of dual system buildings. As a thumb rule, in the typical construction practice, the shear wall area ratio in RC dual system buildings with 5 - 20 storey is kept usually around 1 % [8, 9]. This value is majorly based on the post-earthquake survey reports.

Studies have been carried out to determine the optimum shear wall area ratio for the adequate performance of the building. Wallace and Moehle [10] evaluated the response of frame-shear wall buildings in the United States. They indicated that roof drifts in case of buildings with a shear wall area ratio of more than 1.5 % and a wall aspect ratio of 5 or less (in the direction of loading) is less than 1.0 %, under strong ground motions. Wallace [8] proposed an analytical approach to study the effect of variation of roof drift ratio, spectral displacement, extreme fibre strain and confinement requirement at boundary elements, with shear wall area ratio in RC buildings. The results of a parametric study carried out by Yakut and Soydas [11] on low to mid-rise RC buildings designed according to the Turkish seismic code indicate that the increase in the shear wall area ratio beyond 2.0 - 2.5 % does not result in significant improvement in the seismic performance of buildings. Burak and Comlekoglu [12] evaluated the effect of shear wall area ratio on the IS drift, roof drift and base shear response of RC buildings. They concluded that a minimum of 1 % of the shear wall area ratio is sufficient to control the drift requirements in mid-rise (5 - 8 storey) RC buildings. Also, the increase in shear wall area ratio beyond 1.5 % does not significantly improve the seismic performance of buildings.

In the past two decades, significant development has been made in the performance-based seismic design procedure, and it has been accepted that collapse risk is a crucial criterion for the seismic design and evaluation of buildings [2, 13, 14]. FEMA P695 [13] provides a framework for evaluation of current code-approved structural systems for their ability to achieve intended 'life-safety' seismic performance objective. This study evaluates the seismic behaviour and performance of Indian code conforming high-rise RC dual system buildings in terms of its collapse capacity and seismic fragility, following FEMA P695 methodology. For this, a 15-storey archetype building utilizing planar RC walls and moment frame, as the lateral force-resisting system is designed in line with current Indian seismic design codes [15, 16]. The area of shear walls and their configuration is varied on the same building plan to study their effect on the collapse capacity and collapse margin safety. The performance of the building is evaluated using non-linear static and non-linear time-history analyses. The probability of collapse of the buildings is considered to estimate the optimum area of shear walls as a fraction of the plan area. The results are then analyzed and discussed, aimed at establishing the adequacy and identifying the limitations of the seismic design approaches for dual system buildings in the Indian code.

2. Code design provisions

The shear walls are primarily subjected to mainly shear or flexure action depending upon the aspect ratio of the wall, along with axial loads. The behaviour of squat wall is generally governed by shear whereas for slender wall it is governed by flexure. In order to ensure that the shear will not affect the ductile behaviour of the shear wall, its strength should be controlled in order to avoid the brittle mode (shear) and should be designed according to the capacity design principle. The principle of capacity design is to control and eventually to increase the capacity of a structural member relatively to an undesirable mode of failure, in order to prevent it during an earthquake. This capacity design increases the hysteresis energy dissipation capacity of walls during earthquakes. The current IS 13920 [16] does not give provisions for the capacity design of shear walls.

IS 13920 [16] provides guidelines to design shear walls. The minimum thickness of any part of the shear wall section should be greater than 150mm. The minimum value of horizontal and vertical reinforcement in the shear wall is given depending upon the dominant shear/bending behaviour (squat, intermediate or slender). The diameter of the reinforcing bars should not exceed one-tenth of the thickness of the wall. The reinforcement in shear walls should be provided in two layers if the thickness of the wall exceeds 200mm or factored stress value exceeds 0.25 times the square root of the characteristic strength of concrete. The maximum spacing of both horizontal and vertical reinforcement shall not exceed the smaller of $3t_w$, $l_w/5$, and 450mm; where l_w is the horizontal length of the wall, and t_w is the thickness of wall web. The elements which are provided along the wall edges are termed as 'Boundary Elements'. These elements are strengthened with vertical and transverse reinforcement. These are designed as a short column. Whenever the extreme fibre compressive stress in the wall exceeds $0.2f_{ck}$, the boundary elements shall be provided. The boundary elements may be discontinued wherever compressive stress is less than $0.15f_{ck}$. Special confining reinforcement should be provided throughout the height of the boundary element. If boundary elements are not provided, a minimum of 4 vertical bars of 12mm diameters should be provided in two layers at the ends of the walls in a distance not exceeding twice the thickness of the shear wall. The boundary elements are not required if the entire wall section is provided with special confining reinforcement.

3. Numerical study

The building models are generated based on the database of a pilot survey in the 'NOIDA', a township in the National Capital Region (NCR) of India [17]. The buildings are 15-storey high with a constant storey height of 3.3 m and plinth height of 1.5 m. Three dual system building models with the same floor plan but different shear wall area and their arrangement are considered, as shown in Fig. 1. The ratio of shear wall to floor plan area in these buildings is maintained around (approximately) 0.45, 0.90 and 1.35 %, respectively. These models are thus referred to as SW_R = 0.45, SW_R = 0.90 and SW_R = 1.35, respectively. Table 1 presents the general properties of the buildings considered. Two models of dual system building with shear wall area ratio of about 0.45 % are considered: one with columns and beams of the moment frame having strong column-weak beam ratio of more than 1.4 and another one without strong column-weak beam design, (hereafter, referred as SW_R = 0.45_{SCWB} and SW_R = 0.45 respectively) to study its effect on the seismic performance of the building. The other models (SW_R = 0.90 and SW_R = 1.35), where the influence of shear wall in the base shear contribution is even larger (more than 80 %), the moment frame is designed without strong column-weak beam criterion.

The buildings are designed for both gravity and seismic load actions. The RC design is carried out using the recommendations of IS 456 [18] and ductile detailing is performed following the recommendations of IS 13920 [16]. The concrete of characteristic cube strength as 40 N/mm^2 and reinforcing steel of yield strength as 500 N/mm^2 are used for design. The dead and imposed load are applied following IS 875 (Part 1) [19] and IS 875 (Part 2) [20], respectively. The buildings are considered to be situated on a rock site (soil type I) in the highest seismic zone (Zone V) of India (with an effective peak ground acceleration (EPGA) at MCE level (2% in 50 years) as 1.50 × 0.36g). The moment frame in the dual system buildings is designed as a special moment-resisting frame (SMRF) with ductile detailing, and capacity design provisions [16]. The moment frame is designed to resist a minimum of 25% of the base shear according to the provisions of IS 1893 [15] for the design of dual system buildings. The reinforcement in the beam and columns is maintained in the range of 0.2 - 1.50 % and 0.8 - 3.5 %, respectively. The planar shear wall is designed using the provisions of IS 13920 [16]. The vertical reinforcement in shear walls is provided in the range of 0.25 - 1.50 %. Adequate horizontal reinforcement is provided to avoid brittle shear failure and boundary elements are confined. The estimated member sizes for each building are presented in Table 2. The effective cracked section properties of structural members are considered from ASCE 41-17 [21] guidelines. The P-delta effects are accounted for in the analysis and design of the buildings.

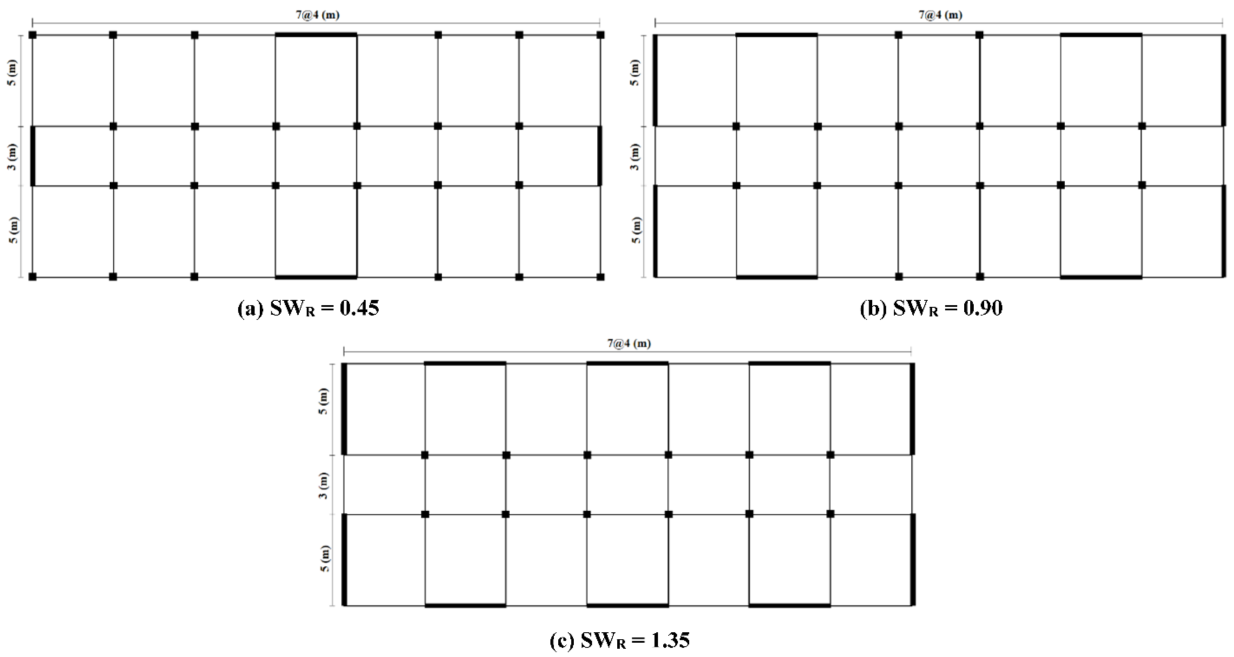

Fig. 1 – Typical storey plan of building models.

Table 1 – Design properties of the building models.

Building model	Period of vibration T^a (in sec)	Shear wall area ratio (%)		Column area ratio (%)	Design base shear coefficient (V/W)	Base shear contribution
		X	Y			Shear wall
$SW_R = 0.45_{SCWB}$	4.29	0.44	0.41	1.34	0.012	70 - 75%
$SW_R = 0.45$	4.29	0.44	0.41	1.34	0.012	70 - 75%
$SW_R = 0.90$	3.28	0.88	0.82	0.89	0.017	85 - 90%
$SW_R = 1.35$	2.93	1.32	1.37	0.67	0.020	90 - 95%

[a] Average translational period of vibration of the building.

3.1 Building modelling

Three-dimensional (3D) models of the considered buildings are created in the structural analysis and design software for building: ETABS [22]. Frame elements are used to model the beams and columns. The slab is modelled as a rigid diaphragm. The shear walls are modelled using the wide-column analogy. In this analogy, the shear wall is modelled as an equivalent column at the centroid of the shear wall section. Finite size of beam-column and beam-shear wall joints is also considered in modelling. The seismic design of the buildings is carried using the mode-superposition (response spectrum) method with a response reduction factor of 5 according to IS 1893 [15] guidelines. The average period of the building (mean of the fundamental periods of the building in the two directions) and design base shear are shown in Table 1.

Table 2 – Estimated member sizes of the considered building models.

Building Model	Beams	Columns	Shear Wall	
			X	Y
$SW_R = 0.45_{SCWB}$	300 x 400	450 × 450 / 400 × 400 / 350 ×350 / 300 × 300	4000 × 200	3000 × 250
$SW_R = 0.45$	300 x 400	450 × 450 / 400 × 400 / 350 ×350 / 300 × 300	4000 × 200	3000 × 250
$SW_R = 1.00$	300 x 400	450 × 450 / 400 × 400 / 350 ×350 / 300 × 300	4000 × 200	5000 × 150
$SW_R = 1.50$	300 x 400	450 × 450 / 400 × 400 / 350 ×350 / 300 × 300	4000 × 200	5000 × 250

Note: All dimensions are in mm.

Lumped plasticity model is used to define the non-linearity in the elements. Uniaxial moment-rotation (M3) plastic hinges are assigned at both ends of the beams (at a relative distance of 0.05 and 0.95). The backbone curves (i.e., force-deformation envelopes of beams) and the acceptable deformation limits for various performance levels (i.e., IO, LS and CP) are obtained from ASCE 41-17 [21]. The cyclic properties of the moment-rotation hinges are modelled using energy-based degrading hysteresis model [22]. The details of the model can be found in the CSI manual [22]. The properties of the model (f_1, f_2 and s) for ductile beams are taken from Surana et al. [23]. The non-linearity in columns and shear-walls is modelled using the fibre-hinge model. The fibre-hinges are assigned at the two ends of the columns, located at the midpoint of the plastic hinge length measured from the face of the connecting element. In case of shear wall, the fibre-hinge is defined only at the base of the shear wall, located at the centroid of the plastic hinge length measured from the base. At each plastic hinge location, the section is discretised into fibres for confined and unconfined concrete and one steel fibre per reinforcing bar. Plastic hinge length for columns is taken as half of the maximum horizontal dimension of the column. Plastic hinge length for the shear wall is obtained from the empirical relation given by Priestley et al. [24]. Mander's model [25] is used for defining the stress-strain relationship for confined and unconfined concrete. The stress-strain curve of the steel reinforcing bar is assumed as a bilinear elastic-plastic model with kinematic strain-hardening. The cyclic deterioration effects have been incorporated in the material properties using energy-based hysteresis model. The parameters for this hysteresis model have been calibrated with the experimental results on columns [26] and shear-walls [27] available in the literature. Results here are not shown for brevity. The strain in the extreme fibre is used to indicate the performance level (IO, LS and CP) in case of fibre-hinge model. These limit values of strains corresponding to different performance levels are taken from Turkish seismic code [28].

3.2 Non-linear analysis

The seismic performance of the buildings is estimated using non-linear static analysis (NSA) and non-linear dynamic analysis (NLTHA). In non-linear static analysis (NSA), the capacity curve (popularly known as 'pushover curve') of the building, which represents the plot between the base shear and roof displacement, is obtained under an assumed distribution of lateral load. The lateral load pattern is assumed to be proportional to the fundamental mode of the building in the two directions. The pushover analysis is carried out to determine the useful information on the yield strength, failure mechanism, overstrength and ductility factors of the buildings. The median collapse capacity and seismic fragility of the building models are determined using

NLTHA. In NLTHA, a suite of ground motion is selected, and each ground motion is scaled with increasing intensity to study overall behaviour of buildings, i.e., elastic to inelastic level and finally till global instability level causing building collapse. This method is known as Incremental dynamic analysis (IDA) [29]. The selection of ground motions for the performance assessment of the buildings is an important task due to the wide range of seismological and other factors affecting it. In the present study, a set of far-field two-component horizontal ground motions, recommended by FEMA P695 [13], are used for bi-directional IDA. These ground motions are selected taking into account magnitude (M_w), source-to-site distance (R_s), number of records (N), site class, peak ground acceleration (PGA), peak ground velocity (PGV) and the lowest usable frequency (f_l).

The bi-directional IDA is carried by application of two components of the ground motions simultaneously in the two orthogonal directions, resulting in 44 collapse curves for each building. At each incremental intensity measure (IM) of ground motion, the response of the building is recorded in terms of maximum IS drift (θ_{max}). The IM used is $S_{a,avg}$ (0.2T-3T, 5%), which represents the geometric mean of spectral accelerations (5% damped) of the ground motion in the interval 0.2T - 3T, where T is the average of the period of vibration of the building. The use of $S_{a,avg}$ as IM includes the effect of higher modes and period elongation during non-linear analyses. The scaling of each component is done by the geometric mean of their spectral acceleration. The collapse of the building is defined as the state at which slight increase in IM results in an abrupt rise in the response of the building (maximum IS drift in the present case). A Rayleigh damping model with 5% damping is assigned to the model at the 1st mode period, and the period with cumulative mass participation of 90%. The seismic fragility of the buildings is determined using results from IDA, i.e., median collapse capacity ($S_{a,avg}$ (C)) and record-to-record variability (β_{RTR}), using FEMA P695 [13] methodology. The total variability for fragility analysis is obtained as the sum of square root of modelling variability (β_M) and record-to-record variability (β_{RTR}). The modelling variability takes into consideration the effect of existing construction practices, construction material, the robustness of the analytical model used for design and simulation of collapse. These values are taken from previous study [30].

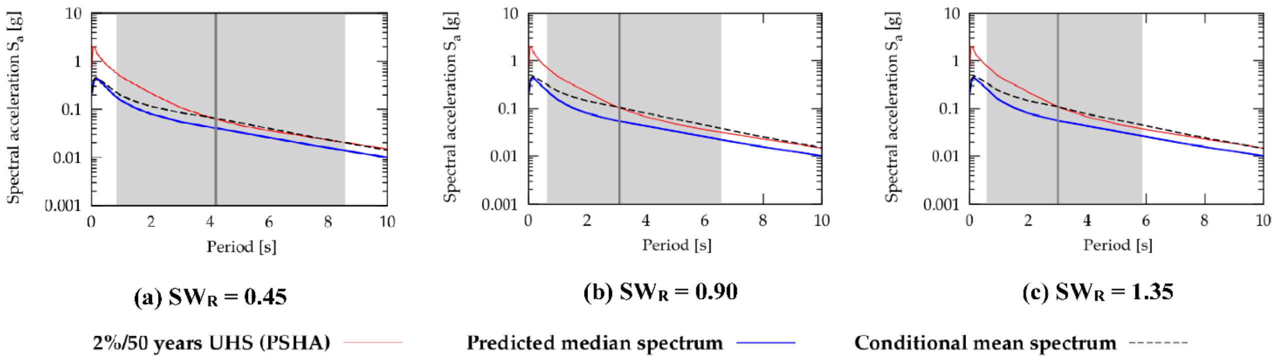

Fig. 2 – Conditional mean spectrum of building models.

For all the buildings, the collapse margin ratio (CMR), and the probability of collapse of the building is computed at DBE (return period = 475 years) and MCE (return period = 2475 years) demand levels. CMR is defined as the ratio of the median collapse capacity ($S_{a,avg}$ (C)) of the building expressed in terms of the spectral acceleration (obtained from IDA in the present study) to the spectral acceleration demand at MCE level ($S_{a,avg}$ (D_{MCE})). To compute the MCE level demand, a site in 'Guwahati' region is selected in Zone V of the Indian sub-continent. The uniform hazard spectrum (UHS) is obtained for the region by performing probabilistic seismic hazard analysis for the site using the OpenQuake [31]. The source models of Indian region developed by Nath and Thingbaijam [32] and already implemented in OpenQuake engine [33] is used for hazard analysis. The UHS is computed in terms of spectral acceleration $S_a(T)$ using the NGA West 2 ground motion prediction equations (GMPE) of Campbell and Bozorgnia [34] and Chiou and Youngs [35]. The values of $S_{a,avg}$ (0.2T–3T, 5%) as IM are then obtained by the computation of conditional mean spectra [36] of the site for each building period, individually. The ε value at the conditioning period is obtained by disaggregation. The value of ε at periods other than the conditioning period is obtained using the cross-correlation coefficients developed for PEER NGA ground motion models by Baker and Jayaram [37]. Fig. 2 shows the computed CMS for considered buildings at MCE demand level.

4. Results and Discussion

Although IDA is employed in this study to assess the seismic performance of the buildings under a suite of selected ground motion records, pushover analysis is first conducted to obtain an initial estimate of the yield strength and to estimate the overstrength and ductility factors of the four considered buildings. Pushover analysis is also helpful for understanding the failure mechanism of the buildings. Fig. 3 presents the static capacity (pushover) curves for the considered buildings in the two orthogonal directions. Table 3 shows the value of yield strength, overstrength and ultimate ductility factor of the considered building models. These values are obtained from the equivalent linearisation of the obtained static capacity curves of the buildings according to the method described in ASCE 41-17 [21]. The yield strength capacities and overstrength obtained for the two-building models ($SW_R = 0.45_{SCWB}$ and $SW_R = 0.45$) are the same, since the failure of the shear wall (rather than moment frame) governs the failure mechanism of the two buildings. It can be observed from Table 3 that the yield capacity increases with increase in SW_R, whereas the ductility and overstrength factors are affected slightly with increase in SW_R.

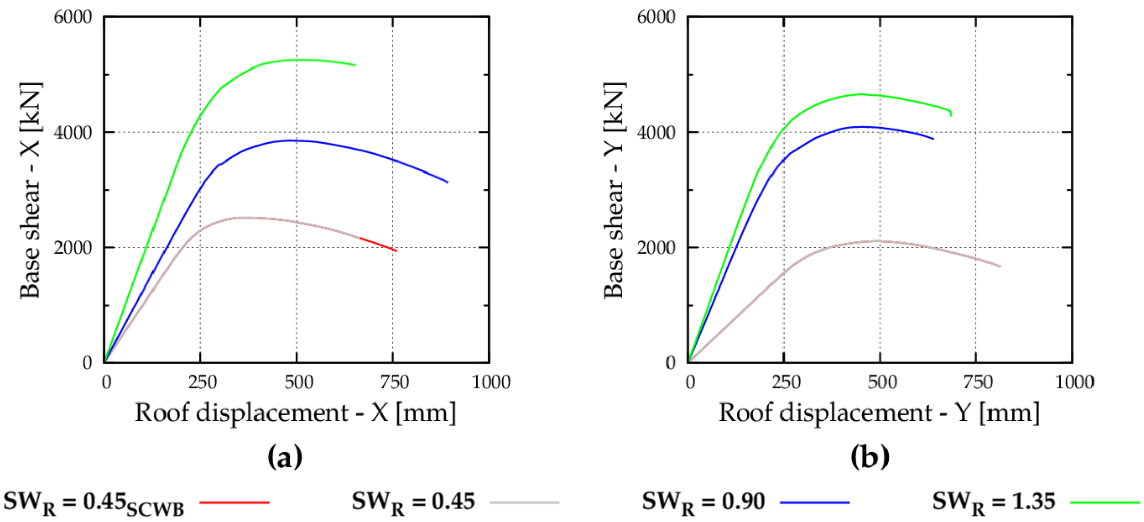

Fig. 3 – Static capacity (pushover) curves of the four building models with different shear wall area ratio. (Please note that the capacity curves for $SW_R = 0.45_{SCWB}$ and $SW_R = 0.45$ are overlapping)

Table 3 – Static capacity curve parameters of the building models.

Building Model	Yield strength (kN)		Overstrength		Ductility	
	V_{yX}	V_{yY}	Ω_X	Ω_Y	μ_X	μ_Y
$SW_R = 0.45_{SCWB}$	2625	2239	2.69	2.29	3.27	2.96
$SW_R = 0.45$	2625	2239	2.69	2.29	2.99	2.96
$SW_R = 0.90$	3931	3950	2.74	2.76	2.53	2.87
$SW_R = 1.35$	4991	4560	2.69	2.46	2.54	2.76

The dynamic capacity curves obtained from bi-directional IDA are shown in Fig. 4. The IDA curves are plotted in terms of $S_{a,avg}$ (0.2T-3T, 5%) as IM and the maximum IS drift (θ_{max}) as response parameter. In the figure, the grey lines present the dynamic capacity curve of individual ground motions (44 curves), the red line displays the median (50th percentile) of all the capacity curve, and green and blue present the 16th percentile and 84th percentile capacity curves, respectively. It can be observed that the median collapse capacity of dual system building increases with the increase in the area of shear walls. This increase in collapse capacity of buildings is due to the design of buildings with more shear wall area for higher base shear coefficients. As the stiffness of the building increases with increase in shear wall area, its period reduces and hence due to the shape of the design response spectrum, the base shear coefficient used for design, increases. The median collapse capacity for dual system building designed with and without strong column-weak beam phenomenon in the moment frame (SW_R = 0.45_{SCWB} and SW_R = 0.45) is almost the same. The results of SW_R = 0.45_{SCWB} are not presented here for brevity. Hence, for dual system buildings where shear wall contributes more than 70% to base shear distribution, the design of moment frame for the strong column-weak beam is not required, since the failure of shear wall mainly governs the collapse mechanism.

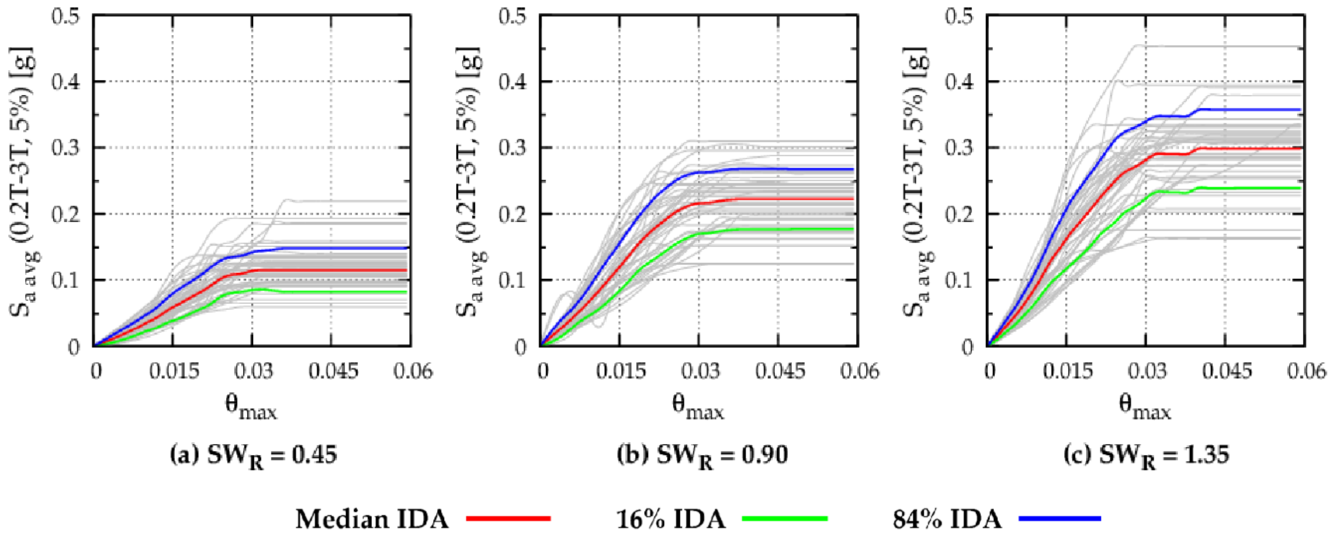

Fig. 4 – Dynamic capacity (IDA) curves of considered building models.

The results from IDA are processed to obtain the value of record-to-record variability. The variability parameters used in this study are presented in Table 4. The collapse margin ratio (CMR) obtained for the considered buildings is shown in Table 5. CMR is utilised to serve as a parameter for comparing the relative collapse resistance of building models. FEMA P695 [13] defines acceptable performance in terms of two collapse indicators. Firstly, the computed CMR estimate should be less than the limiting CMR values given in FEMA P695 [13] corresponding to a 10 % probability of collapse. Secondly, the limiting value of the probability of collapse for an archetype group should be less than 10 %. These acceptable limits for 10% collapse probability and based on the total variability corresponding to the collapse of each building model, are also presented in Table 5. It can be seen that the collapse resistance of SW_R = 0.45 is slightly lesser than the acceptable value. However, the collapse resistance increases considerably, when the area of shear wall is increased.

Table 4 – Variability parameters and the probability of collapse of the building models.

Building Model	β_M	β_{RTR}	β_T
SW_R = 0.45_{SCWB}	0.50	0.26	0.56
SW_R = 0.45	0.50	0.26	0.56
SW_R = 0.90	0.50	0.22	0.55
SW_R = 1.35	0.50	0.22	0.54

Note: β_M - modelling variability; β_{RTR} - record-to-record variability; β_T – Total variability.

Table 5 – Collapse margin ratio of the building models.

Building Model	$S_{a,avg}$ **(C)**	$S_{a,avg}$ **(D$_{MCE}$)**	**CMR**	**ACMR value$_{10\%}$**	**Performance Acceptability**
$SW_R = 0.45_{SCWB}$	0.11g	0.06g	2.00	2.05	unsafe
$SW_R = 0.45$	0.11g	0.06g	2.00	2.05	unsafe
$SW_R = 0.90$	0.22g	0.09g	2.40	2.02	safe
$SW_R = 1.35$	0.30g	0.10g	3.00	2.00	safe

Note: $S_{a,avg}$ (C) and $S_{a,avg}$ (D_{MCE}) are the median collapse capacity obtained from IDA and seismic demand level at MCE level, respectively; ACMR value$_{10\%}$ is acceptable collapse margin ratio value corresponding to 10 % probability of collapse.

The fragility curves for buildings (SW_R = 0.45, 1.0 and 1.35) are shown in Fig. 5. The fragility curve is the plot of the probability of collapse P[$C|S_{a,avg}$] of the buildings with respect to the IM $S_{a,avg}$ (0.2T-3T, 5%). This IM in the present case is normalised with respect to the seismic demand at MCE i.e., $S_{a,avg}$ (D_{MCE}). Such normalisation of the IM allows direct comparison of the fragility curves for different buildings having different periods and different $S_a(T_1)$. The probability of buildings corresponding to DBE and MCE demand level is presented in Table 6. It is observed that the collapse probability of SW_R = 0.45 building is higher than 10 %, which is the limiting value for an archetype group. In contrast, the probability of collapse of SW_R = 0.90 and 1.35 buildings are well within the acceptable range. The significant reduction in the collapse probability between 0.45, 0.90 and 1.35 indicates that shear wall area ratio of around 0.45 % should not be used in design of dual system buildings. Shear wall area ratio of 0.90 and 1.35 provide acceptable seismic performance of dual system buildings.

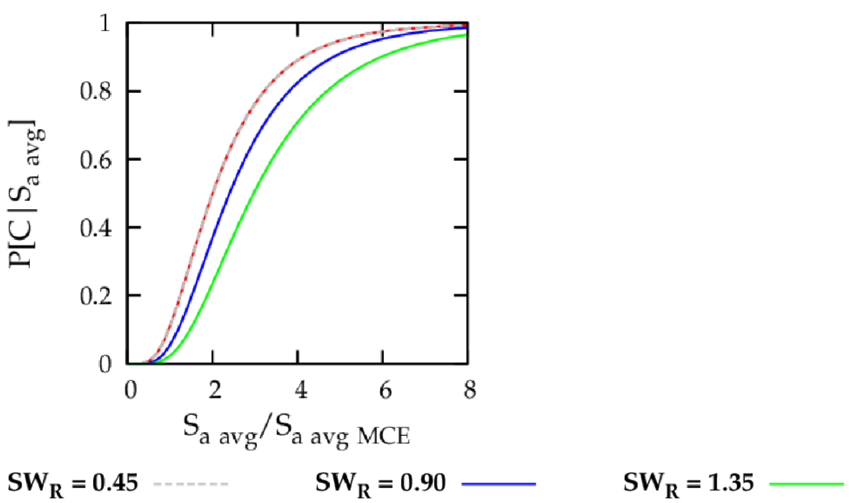

Fig. 5 – Fragility curves of building models.

Table 6 – Probability of collapse of the building models.

Building Model	**P[$C|S_{a,avg}$]$_{DBE}$**	**P[$C|S_{a,avg}$]$_{MCE}$**	**Performance Acceptability**
$SW_R = 0.45_{SCWB}$	1 %	11 %	unsafe
$SW_R = 0.45$	1 %	11 %	unsafe
$SW_R = 0.90$	0 %	5 %	safe
$SW_R = 1.35$	0 %	2 %	safe

Note: P[$C|S_{a,avg}$]$_{DBE}$ - probability of collapse at DBE for a given $S_{a,avg}$; P[$C|S_{a,avg}$]$_{MCE}$ - probability of collapse at MCE for a given $S_{a,avg}$.

5. Conclusions

This paper presents evaluation of seismic performance and collapse fragility of a 15-storey RC dual system building that utilizes RC shear walls designed according to current Indian standard provisions. Structural design is conducted using code-based design response spectrum corresponding to highest seismic zone (Zone V) of Indian sub-continent, while performance assessment is done for a site in Zone V Guwahati region for DBE (10 % in 50 years) and MCE demand level (2 % in 50 years). The effect of different shear wall area to floor plan area ratio and overall structural behaviour with increasing shear wall area ratios is studied using pushover analyses and bi-directional incremental dynamic analyses. In the absence of the capacity design provisions for shear wall in Indian standard codes, horizontal reinforcement are provided throughout the length of the shear wall to prevent shear failure. Non-linear modelling approach for structural members used in this study is calibrated using experimental tests results. Comparison of collapse capacity of different building models show that there is increase in the seismic collapse capacity of the buildings with increase in shear wall area. Also, for buildings in which contribution of shear wall to the lateral force resistance is more 70 %, the strong column-weak design in moment frame need not to be followed. The collapse mechanism of the buildings is seen to be governed majorly by the failure of shear wall at the base and beam elements. Seismic fragility analyses showed that performance of the building with shear wall area ratio around 0.45 % is not acceptable. Its probability of collapse is greater than 10 % and has unacceptable collapse safety margin ratio at MCE demand level. For acceptable performance of the dual system building, a minimum shear wall to plan area ratio of around 0.9 % should be provided. This ratio can be further increased to 1.35 % to fulfil the collapse level acceptance criteria.

6. References

[1] Paulay T, Priestley MJN (1992): Seismic Design of Reinforced Concrete and Masonry Buildings. John Wiley & Sons, Inc., New York, USA.

[2] ASCE 7-16 (2016): ASCE/SEI Standard 7-16, Minimum design loads for buildings and other structures. American Society of Civil Engineers, Reston, Virginia, United States.

[3] Fintel M (1995): Performance of buildings with shear walls in earthquakes of the last thirty years. *PCI Journal*, **40** (3), 62-80.

[4] Riddell R, Wood SL, and De La Llera JC (1987): The 1985 Chile earthquake, structural characteristics and damage statistics for the building inventory in Vina del Mar. *Structural Research Series No. 534*, University of Illinois, Urbana, IL.

[5] Doğangün A (2004): Performance of reinforced concrete buildings during the May 1, 2003 Bingöl earthquake in Turkey. Engineering Structures, **26** (6), 841-856.

[6] Almazan J (2010): Performance of aseismic structures during the Maule's earthquake and its possible effect on the seismic codes in Chile. *South American Magazine of Structural Engineering*, **7** (2), 4-28.

[7] Westenenk B, De la Llera JC, Jünemann R, Hube MA, Besa JJ, Lüders C, Inaudi, JA, Riddell R, Jordán R (2013): Analysis and interpretation of the seismic response of RC buildings in Concepción during the February 27, 2010, Chile earthquake. *Bulletin of Earthquake Engineering*, **11** (1), 69-91.

[8] Wallace JW (1994): New methodology for seismic design of RC shear walls. *Journal of Structural Engineering*, **120** (3), 863-884.

[9] Tekel H (2006): Evaluation of usage of 1% shear wall in reinforced concrete structures. *Turkish Eng. News*, **444** (2006/4–5), 57-63.

[10] Wallace JW, Moehle JP (1992): Ductility and detailing requirements of bearing wall buildings. *Journal of Structural Engineering*, **118** (6), 1625-1644.

[11] Yakut A, Soydas O (2010): Evaluation of shear wall indexes for RC buildings. *9th US National and 10th Canadian Conference on Earthquake Engineering*, Toronto, Ontario, Canada

[12] Burak B, Comlekoglu HG (2012): Effect of shear wall area to floor area ratio on the seismic behavior of reinforced concrete buildings. *Journal of Structural Engineering*, **139** (11), 1928-1937.

[13] FEMA (2009): Quantification of seismic performance factors. *FEMA P695*, Federal Emergency Management Agency, Washington, D.C., United States.

[14] FEMA (2018): Seismic performance assessment of buildings - Volume 1 methodology. *FEMA P58*, Federal Emergency Management Agency, Washington, D.C., United States.

[15] BIS (2016): Indian standard criteria for earthquake resistant design of structures, Part 1: General provisions and buildings (Sixth revision) IS 1893 Part 1. Bureau of Indian Standards, New Delhi.

[16] BIS (2016): Ductile design and detailing of reinforced concrete structures subjected to seismic forces – Code of practice – IS 13920. Bureau of Indian Standards, New Delhi, India.

[17] DEQ (2009): Seismic Vulnerability of Multistorey Buildings in Noida. Department of Earthquake Engineering, Indian Institute of Technology Roorkee, India, 1-118.

[18] BIS (2000): Indian Standard – Plain and reinforced concrete, Code of practice, IS 456. Bureau of Indian Standards, New Delhi, India.

[19] BIS (1987a): Indian standard – Code of practice for design loads (other than earthquake) for buildings and structures (Dead loads), IS 875 Part 1. New Delhi, India.

[20] BIS (1987b): Indian standard – Code of practice for design loads (other than earthquake) for buildings and structures (Live Loads), IS 875 Part 2. Bureau of Indian Standards, New Delhi, India.

[21] ASCE 41-17 (2017): ASCE/SEI Standard 41-17, Seismic evaluation and retrofit of existing buildings. American Society of Civil Engineers, Reston, Virginia, United States.

[22] CSI (2016): ETABS 2016 Integrated building design software. Version 16.2.1, Computers and Structures Inc., Berkeley, U.S.A.

[23] Surana M, Singh Y, Lang DH (2018): Seismic Characterization and Vulnerability of Building Stock in Hilly Regions. *Natural Hazards Review*, **19** (1), 04017024.

[24] Priestley MJN, Calvi GM, Kowalsky MJ (2007): Displacement Based Seismic Design of Structures. IUSS Press, Pavia, Italy.

[25] Mander JB, Priestley MJ, Park R (1988): Theoretical stress-strain model for confined concrete. *Journal of Structural Engineering*, **114** (8), 1804-1826.

[26] Rodrigues H, Dias Arede A, Varum H, Costa A (2013): Experimental evaluation of rectangular reinforced concrete column behaviour under biaxial cyclic loading. *Earthquake Engineering and Structural Dynamics*, **42** (2), 239-259.

[27] Dazio A, Beyer K, Bachmann H (2009): Quasi-static cyclic tests and plastic hinge analysis of RC structural walls. *Engineering Structures*, **31** (7), 1556-1571.

[28] Turkish Seismic Code (2007): Specifications for Buildings to be built in Seismic Zones. Ministry of Public Work and Settlement Government of Republic of Turkey, Ankara, Turkey.

[29] Vamvatsikos D, Cornell CA (2002): Incremental dynamic analysis. *Earthquake Engineering and Structural Dynamics*, **31** (3), 491-514.

[30] Liel AB, Haselton CB, Deierlein GG, Baker JW (2009): Incorporating modelling uncertainties in the assessment of seismic collapse risk of buildings, *Structural Safety*, **31** (2), 197-211.

[31] Silva V, Crowley H, Pagani M, Monelli D, Pinho R (2014): Development of the OpenQuake engine, the Global Earthquake Model's open-source software for seismic risk assessment. *Natural Hazards*, **72** (3), 1409–1427

[32] Nath SK, Thingbaijam KKS (2012): Probabilistic seismic hazard assessment of India. *Seismological Research Letters*, **83** (1), 135-149.

[33] *About the IndiaOpenPSHA*, retrieved from https://github.com/nackerley/indian-subcontinent-psha (last accessed Jan 2020).

[34] Campbell KW, Bozorgnia Y (2014): NGA west 2 ground motion model for the average horizontal components of PGA, PGV, and 5% damped linear acceleration response spectra. *Earthquake Spectra*, **30** (3), 1087-1115.

[35] Chiou BSJ, Youngs RR (2014): Update of the Chiou and Youngs NGA model for the average horizontal component of peak ground motion and response spectra. *Earthquake Spectra*, **30** (3), 1117-1153.

[36] Baker JW (2011): Conditional mean spectrum: Tool for ground-motion selection. *Journal of Structural Engineering*, ASCE, **137** (3), 322-331.

[37] Baker JW, Jayaram N (2008): Correlation of spectral acceleration values from NGA ground motion models. *Earthquake Spectra*, **24** (1), 299-317.