PERFORMANCE EVALUATION USING FRAGILITY ANALYSIS OF RC FRAME - WALL STRUCTURES

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Abstract

A frame-wall structure provides resistance to lateral loading by a combination of shear walls and rigid frames. Due to the difficulties in predicting earthquakes and its random nature, probabilistic analysis is proposed in analyzing structural seismic responses. Fragility curve represents a continuous relationship between a seismic intensity measure and the probability that the structure will reach or exceed a predefined damage state. The fragility analysis is carried out using lognormal distribution of clouds of responses obtained using Incremental Dynamic Analysis (IDA). In the present paper, 15, 18, 22 and 26 storeys RC moment resisting frame-wall structure are analysed for seismic zone IV. The structures are assumed to be resting on hard soil and are analysed using ETABS-2016 and designed as per IS code provisions. Geometrical configuration of the structure is considered as per IS 16700:2017. The performance evaluation of above frames is done using SeismoStruct software for set of 11 recorded ground motions of past Indian earthquake varying in range of magnitude from 5.6 to 7.8. For this study, limit states Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) are considered as the performance criteria referred from FEMA 356. From the fragility curves, it is observed that there is negligible probability of collapse for spectral acceleration corresponding to Design Basis Earthquake. Also, probability of exceedance increases as the number of storey increases at given level of spectral acceleration. This is due to reduction in median collapse capacity of building.

Keywords: Frame-Wall Structure, IS 16700, Incremental Dynamic Analysis, Fragility Analysis, Performance Evaluation.

I. Introduction

One of the most prevailing system in medium to high rise structure is wall-frame system. Shear walls are used in high-rise buildings to resist lateral loads. Often they supplement frames, which if unassisted, could possibly not efficiently withstand the lateral loads. Majority of the buildings, therefore, are frame-shear wall. The distributing lateral loads is based on the interaction behaviour of the frame-shear wall system.

As the independent behaviour of frame and shear wall under seismic action are diverse, the combined effect is beneficial in context to deformation limits and economy. When these two are held together by floor slabs or beams, the cumulative pattern of deformation is different. Likewise, the interacting forces vary in magnitude and direction along the height of the structure as shown in Fig. 1.

The shear wall primarily responds by flexure as a cantilever, whereas, the frame deflects in a shear mode. Compatibility of the two, assuming that the beams or floor slabs have sufficient in-plane stiffness to produce equal lateral displacements at the floor level, generates interaction between the wall and frame. The parabolic sway of the shear walls and the linear drift of the moment frame, leads to enhanced stiffness because the walls are restrained by the frames at the upper levels while at the lower levels the frames are restrained by the walls. However, a closely spaced frame with deep beams tends respond predominantly in a flexure mode. Correspondingly, a shear wall weakened by large openings acts more like a frame by deflecting in a shear mode. Hence, the combined structural action depends on the relative stringency of the two, and their modes of deformation. [11]

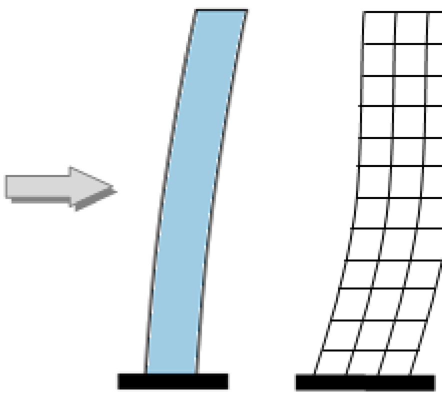

Figure 1: *ShearWall and Frame Deformation Shape under Lateral Load*

The behaviour of reinforced concrete structures under the effect of ground motions has always been a subject of investigation, especially in seismic region. Meanwhile, the damage to buildings from recent earthquakes has emphasized the need for risk assessment of existing building standard to estimate the potential damage from future earthquakes. Seismic actions are challenging to predict and hence, probabilistic analysis is offered for vulnerability assessment of building responses. In order to accurately capture the nonlinear seismic response of a structure, complex analysis methods and material models are required. Incremental Dynamic Analysis (IDA), an extension of nonlinear time history analysis, is a parametric method of analysis which predicts complete structural responses and performances. A structural model is subjected to a suite of ground motion records and the intensity of these ground motions are monotonically scaled. Plotting of Intensity Measurement (IM) of the scaled ground motions and Damage Measurement (DM) is termed IDA Curve.

Vulnerability evaluations of buildings are normally carried out for judging the requirement for consolidation of vital members against later earthquakes. The best way to achieve such assessments is by Fragility curves. It demonstrates the conditional probability of response of a structure that it may exceed the performance limit at a given ground motion intensity. This approach is beneficial for damage and loss estimation, and disaster response planning. Out of many existing approaches, here fragility analysis is carried out using clouds of responses obtained from IDA using lognormal distribution of median and dispersion parameters.

Using IS code [4], [7] provisions, analysis and design of RC Frame-Wall Structure with 15, 18, 22 and 26 Storey is carried out. The frames are assumed to be resting on hard soil and lying in seismic Zone IV. IDA with monotonic scaling till numerical non-convergence of above frames is done using SeismoStruct software for set of 11 recorded ground motions of past Indian earthquake. Using IDA results, considering performance criteria as per FEMA 356[3], fragility curves are obtained for probabilistic assessment of vulnerability.

I. Fragility Analysis

In IDA, the structural model is subjected to set of seismic ground motion records whose intensity are increased monotonically using scale factors. It continues to proliferate, ranging the structural responses to go from elastic to the nonlinear till collapse. Moreover, if both Capacity (C) and Demand (D) of a structure are subjected to time variation, it is difficult to predict safety of structure. At certain time t, Safety (S) can be expressed as St = Ct - Dt where if demand exceeds capacity, it indicates a hazardous state. Probability distribution for C and D can be thus carried out for better estimate of safety. Fragility curves are a crucial component of probabilistic seismic risk assessment. Fragility curve (F) defines continuous relationships between a ground motion intensity measure (IM) and the total probability that the specified structure will reach or exceed predefined engineering demand parameter (EDP); they can be expressed as:

$$F = P\,[D > C \mid IM] \qquad (1)$$

Where, D is the demand of the asset class being assessed and C is a specific predefined state of damage. Fragility may be evaluated using either EDP or IM ordinates versus their associated capacities EDPC and IMC, respectively. An IM is a scalar representative of the earthquake damage potential with respect to the specific structure.

Procedure of Developing Fragility Curve Using Incremental Dynamic Analysis:

- Computer model of building is analysed and designed.
- Ground motion records to perform dynamic response history analysis are selected.
- Solve equation of motion at each time step and record the response parameters.
- Use scaling to increase the IM level of the ground motion record, and repeat the process until all the limit states are reached.
- Above steps are repeated for each selected ground motion.
- Cloud of structural response results is plotted.
- Fragility curves are constructed from the set of IM and EDP pairs through an appropriate statistical curve fitting approach.

II. Regression Analysis Using Least Square Formulation:

As the Monte Carlo simulation (MCS) mostly relates to its high computational cost, for such cases, seismic fragility can be computed based on data from cloud. For cloud analysis, regression is required to achieve a continuous representation of the distribution of EDP∣IM for all IM levels of interest, which is generally performed using the power-law approximation:

$$EDP(IM) = aIM^b\,\varepsilon \qquad (2)$$

ε = lognormal random variable, a and b = constants from trend line.

Probability of exceeding a certain damage state given the value the earthquake intensity measure can be expressed as:

$$P(D > C|IM) = \Phi\left(\frac{\ln(aIM^b) - \ln EDP_{C,50\%}}{\beta_{EDP(IM),tot}}\right) \qquad (3)$$

$$= \Phi\left(\frac{\ln a + b\ln IM - \ln EDP_{C,50\%}}{\beta_{EDP(IM),tot}}\right) \qquad (4)$$

$$= \Phi\left(\frac{\ln IM - \ln\left[\left(\frac{EDP_{C,50\%}}{a}\right)^{1/b}\right]}{\beta_{EDP(IM),tot}/b}\right) \qquad (5)$$

$$= \Phi\left(\frac{\ln IM - \ln IM_{C,50\%}}{\beta_{IM,tot}}\right) \qquad (6)$$

Where,

$IM_{C,50\%}$ = Median ground motion intensity (IM)

$\beta_{IM,tot}$ = Associated dispersion of intensity measure (IM) on the EDP capacity

= $\frac{STDEV\ [\ln(\varepsilon)]}{b}$

Φ = Standard normal cumulative distribution function

In this study, $\beta_{EDP(IM)}$ uncertainty of ground motion only is taken into consideration in order to understand the behaviour of structures subject to ground acceleration variations considering material uncertainty as null.

II. Analysis and Design of Structure

All buildings of 15, 18, 22, 26 storey are analysed using ETABS 2016 software and designed as per provisions of IS codes. Geometrical configuration of RC moment resisting frame with shear wall is considered as per IS 16700:2017 [5]. For all the cases, maximum plan aspect ratio (L/B) does not exceed 5.0. Further, the maximum building height does not exceed 100 m as per criteria of structures located in Zone IV. The slenderness ratio of height (H) to minimum base width (B) does not exceed 8. For design purpose, concrete grade of M40 with reinforcement steel of Fe500 grade are used. Response reduction factor of 5 with importance factor of 1.2 are assumed with structure lying on hard soil. Typical storey height was taken as 3.40 m. Dead load including self-weight of beams, columns, wall and slabs are considered and the imposed load taken as 4.00 kN/m^2. Effective moment of inertia for the design of beam and column/shear wall considered is 0.35 Ig_0 and 0.70 Ig_0 respectively, while that for drift for beam and column/shear wall considered are 0.70 Ig_0 and 0.90 Ig_0 respectively. Typical plan of all structures under study is of 18 m x 36 m as shown in Fig.2.

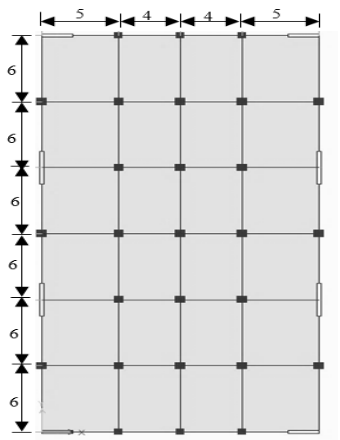

Figure 2: *Typical Geometric Configuration in Plan*

As per IS 16700:2017, when design lateral forces are applied on the building, the maximum inter-storey elastic lateral drift ratio (Δmax/hi) under wind load, shall be curbed to H/500. For earthquake load combinations the drift shall be limited to hi/250.

RC Moment Resisting Frame-Wall Structure with 15, 18, 22 and 26 storey are analyzed and designed for load combinations as per Indian Standards. The design base shear, and modal time period is shown in Table 1.

Table 8 : *Modal Time Period and Design Base Shear*

Storey	Time Period of First Mode (s)	Base Shear, Vbx (kN)
15	3.63	3806.45
18	4.26	3847.24
22	4.64	3897.34
26	5.49	4113.04

Dimensions of beam is computed such that steel reinforcement designed for both moments range from maximum 1.10% to minimum criteria. For column, design steel reinforcement is computed for axial force and bi-axial moments. Dimensions are selected so as to keep maximum percentage of steel up to 3.37%. Shear Wall is designed with boundary elements with maximum 2.54% of steel reinforcement. SAFE 2016 software is used for design of footing as per provisions of IS code. Buildings are considered to be resting on hard soil and safe bearing capacity (SBC) of soil assumed is 350 kN/m^2. Sections of each member are reduced with elevation for optimized reinforcements as per Indian Standards.

I. Nonlinear Modelling

SeismoStruct software is used for performing nonlinear analysis. The concrete model used was proposed by Mandar et al (1988) with both confined and unconfined section properties. Reinforcement steel used is a uniaxial bilinear stress strain model. This simple model is also characterized by its computational efficiency.

Distributed plasticity is chosen for elements idealized to fibre sections with multiple integrating points using Inelastic Force-Based Plastic Hinge Frame type giving forced based formulation. It concentrates such inelasticity within a fixed length of the element. The advantages of such formulation are a reduced analysis time, and full control of the spread of inelasticity. The number of section fibres used in equilibrium computations needs to be defined. In addition, the plastic hinge length needs also to be demarcated. Reinforced concrete rectangular wall section is used to model shear walls with edge sections.

In this research, P-delta effect was considered on the RC columns and shear walls when gravity loads introduce compression axial force and lateral loads perpendicularly applied to the columns and shear walls. Fixed support was adopted for the foundations. Top of the building is free to move both translationally and rotationally having the maximum deformation.

II. Past Earthquake Records

Out of the available ground motion records from database, the following criteria were selected:

Chosen ground motions have a magnitude greater than 5.5. The site condition is classified as rock. The recording is made in the far field at a distance greater than 10 km. Considered earthquakes has Peak Ground Acceleration (PGA) more than 0.10g and Peak Ground Velocity (PGV) more than 10 cm/s.

Ground motion records were recorded in different locations pan India. Response spectrum of such 11 recorded ground motions of past Indian earthquake along with design response spectrum are shown in Fig.3

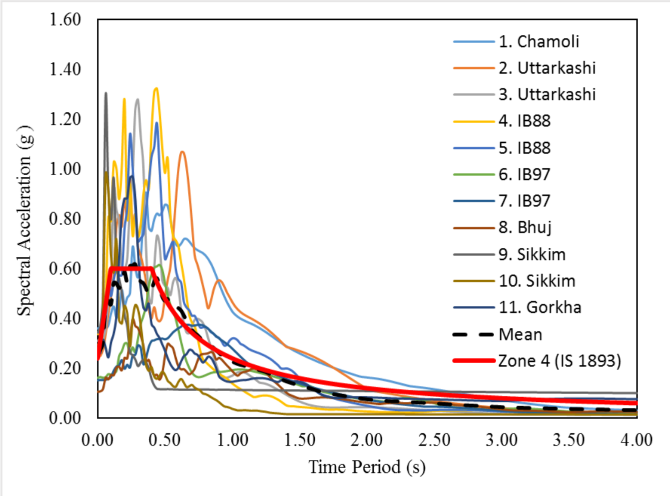

Figure 3: *Ground Motion Response Spectra and Design Spectra*

III. Results and Discussions

After designing the structures under study, nonlinear dynamic analysis is performed using IDA. The results of IDA response are then further used to plot fragility curves to indicate the probability of exceedance of various limit states considered as per FEMA356. Fragility curves are plotted using regression analysis of clouds of response and its lognormal distribution.

I. Clouds of Response

For the purpose of illustration, clouds of response of 15 Storey Frame-Wall structure has been represented. Fig. 4 shows the IDR (%) v/s Sa(g) plot from IDA. Fig. 5(a), 5(b), 5(c) are the logarithmic plots for IO, LS and CP limit states respectively for computation of median and standard deviation parameters for fragility analysis.

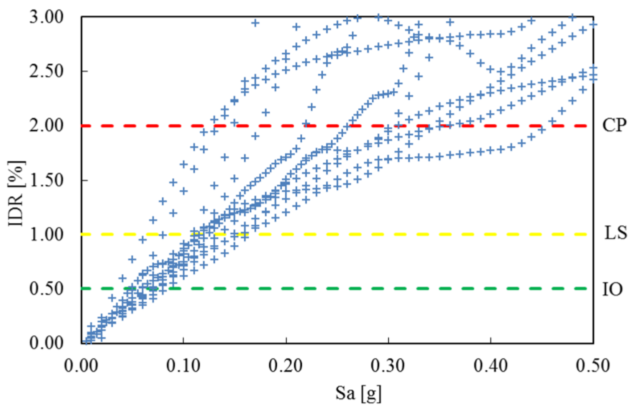

Figure 4: *Plot of IDR(%) v/s Sa(g) from IDA*

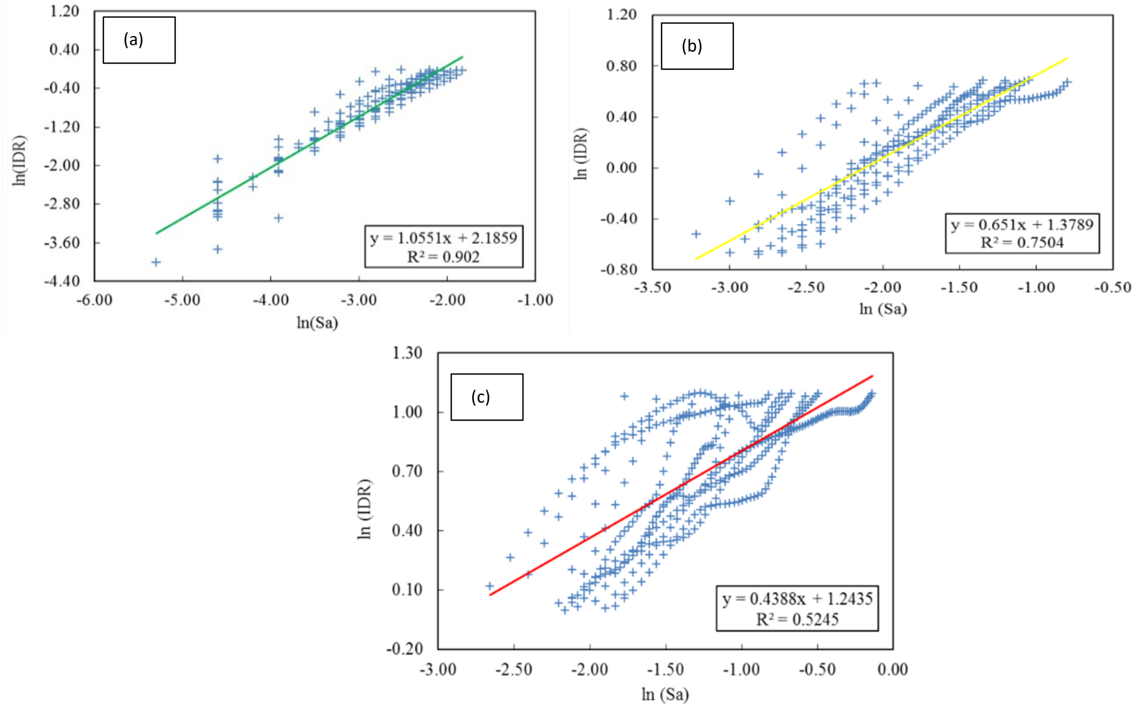

Figure 5: *Clouds of Response of 15 Storey Building at (a) IO (b) LS (c) CP limit states*

II. Fragility Parameters

Using least-square regression procedure, the clouds of response are summarized in form of fragility parameters of median and dispersion. These are as indicated in Table 2.

Table 2: *Fragility Parameters*

Performance Levels	Fragility Functions	Storey			
		15	18	22	26
IO	Median	0.065	0.055	0.051	0.045
	Dispersion	0.246	0.273	0.217	0.202
LS	Median	0.120	0.105	0.091	0.083
	Dispersion	0.292	0.309	0.326	0.350
CP	Median	0.285	0.267	0.247	0.241
	Dispersion	0.483	0.346	0.443	0.474

III. Fragility Curves

After obtaining responses from IDA method, they are arranged in ascending order and as per FEMA356 criteria, they are bifurcated in category of IO, LS, and CP. Using computed parameters of median and dispersion, the fragility curves are plotted applying lognormal distribution. The fragility curves obtained are as shown in Fig.6(a), (b), (c), (d) for 15, 18, 22, 26 stories frame-shear wall structure respectively.

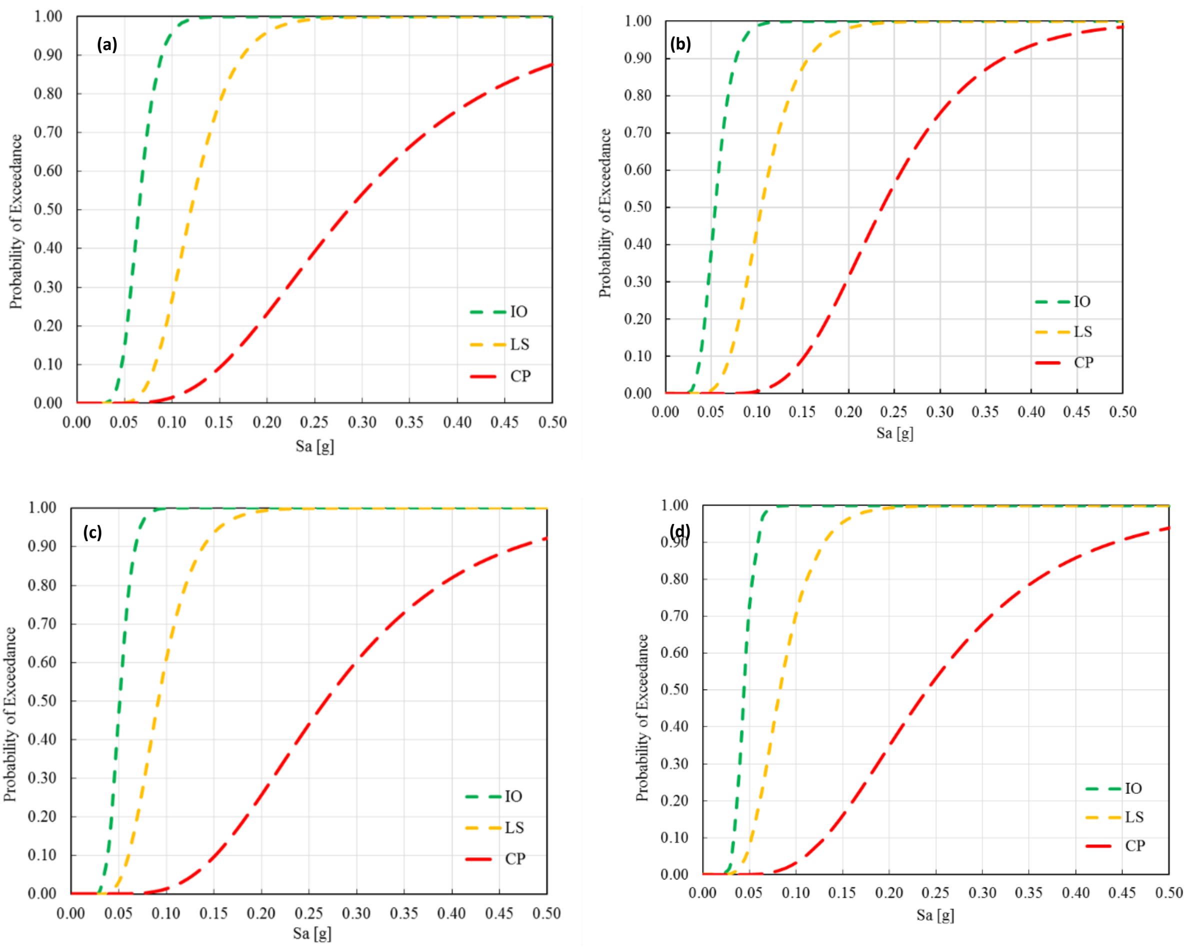

Figure 6: *Fragility Curves for (a) 15, (b) 18, (c) 22, (d) 26 storey buildings*

IV. Summary

It is seen that for spectral acceleration corresponding to Design Basis Earthquake (DBE), all the frame has less than 50% probability of exceedance of Life safety limit state. It is observed that probability of exceedance of Collapse limit state for spectral acceleration corresponding to DBE is negligible.

Spectral acceleration at DBE and Maximum Considered Earthquake (MCE) for each designed building is shown in Table 3.

Table 3: *Spectral Acceleration for DBE and MCE*

Storey	DBE, Sa (g)	MCE, Sa (g)
15	0.111	0.222
18	0.092	0.185
22	0.076	0.151
26	0.064	0.128

Probability of exceedance of DBE and MCE obtained from fragility curve results using nonlinear dynamic analysis is shown in Table 4.

Table 4: *Probability of Exceedance*

Performance Levels	Probability of Exceedance	Storey			
		15	18	22	26
IO	DBE	98%	97%	97%	96%
	MCE	100%	100%	100%	100%
LS	DBE	39%	31%	28%	23%
	MCE	98%	98%	94%	89%
CP	DBE	3%	0%	0%	0%
	MCE	30%	24%	10%	9%

IV. Conclusion

It is observed from the fragility curves that probability of exceedance decreases as the number of storey increases for spectral acceleration corresponding to respective DBE and MCE. This is due to increase in height of buildings resulting in a higher time period and lower spectral acceleration.

From the fragility curves, it is indicated that there is less than 40% probability of exceedance of Life safety limit state and has negligible probability of exceedance at Collapse limit state for spectral acceleration corresponding to respective DBE. Hence, it indicates satisfactory behaviour, though the probability of exceeding IO limit state is above 95%.

It is also observed that probability of exceedance increases as the number of storey increases at given level of spectral acceleration. This is due to reduction in median collapse capacity of building.

References

[1] Asgarian B., Yahyai M., Mirtaheri M., Samani H.R. and Alanjari P. (2010) *Incremental Dynamic Analysis of High-Rise Towers.*, The Structural Design of Tall and Special Buildings, Vol. 19, pp. 922-934.

[2] Bakalis K., Vamvatsikos D. (2018) *Seismic Fragility Functions via Nonlinear Response History Analysis.*, Journal of Structural Engineering, Vol. 144.

[3] Federal Emergency Management Agency, FEMA 356. Prestandard and Commentary for the Seismic Rehabilitation of Buildings. Washington DC.

[4] IS 13920 (2016), Indian Standard for Ductile Design and Detailing of Reinforced Concrete Structures Subjected to Seismic Forces - Code of Practice. Bureau of Indian Standards, New Delhi.

[5] IS 16700 (2017), Indian Standard for Criteria for Structural Safety of Tall Concrete Buildings. Bureau of Indian Standards, New Delhi.

[6] IS 1893 Part 1 (2016), Indian Standard for Criteria for Earthquake Resistant Design of Structures. Bureau of Indian Standards, New Delhi.

[7] IS 456 (2000), Indian Standard for Plain and Reinforced Concrete-Code of Practice. Bureau of Indian Standards, New Delhi.

[8] IS 875 Part 3 (2015), Indian Standard for Design Loads (Other than Earthquake) for Buildings and Structures - Code of Practice. Bureau of Indian Standards, New Delhi.

[9] Kirc M.S. Polat Z. (2006) *Fragility analysis of mid-rise R/C frame buildings*, Engineering Structure-Elsevier., Vol. 28, pp. 1335-1345.

[10] Smith B.S., Coull A. *Tall Building Structures: Analysis and Design.* by John Wiley & Sons, INC.

[11] Taranath B.S. Wind and Earthquake Resistant Buildings. by Marcel Dekker.

[12] Vamvatsikos D., Cornell C.A. (2002), *Incremental Dynamic Analysis.* Earthquake Engineering and Structural Dynamics., Vol. 31, pp. 491-514.