ASPS Conference Proceedings 1: 1719-1725 (2022)

Proceedings of

12th Structural Engineering Convention-An International Event (SEC 2022)

Available at https://asps-journals.com/index.php/acp

Performance Evaluation of RC Frame - Wall Structures Using Incremental Dynamic Analysis

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Paper ID - 230002

Abstract

A structure whose resistance to lateral loading is provided by a combination of shear walls and rigid frames, may be categorized as a frame-wall structure. The behaviour of RC structures under the effect of earthquake loading has always been a subject of investigation. Structural seismic design has been undergoing a major revaluation in recent time, with importance shifting from strength to performance. Nonlinear Time-History Analysis (NLTHA) constitutes the accurate way for simulating response of structures subjected to seismic excitation. Incremental Dynamic Analysis (IDA), involves performing nonlinear dynamic analysis of the structure under a set of ground motion records, each scaled monotonically to several intensity levels. In the present paper, RC moment resisting frame-wall structure with 18, 22 and 26 storeys are analysed for seismic zone IV, resting on hard soil and designed as per IS code provisions. Geometrical configuration of the buildings are considered as per IS 16700:2017. Analysis and design of frames are carried out using ETABS-2016. The performance evaluation of above frames is done using NLTHA and IDA 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 parametric study, the performance criteria as per FEMA 356 for the limit states of Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) are considered. Results obtained from NLTHA are shown in terms of Interstorey Drift Ratio (IDR) profile. The results of NLTHA showed satisfactory performance when evaluated by a set of recorded ground motions of past Indian earthquakes. Individual and summarized IDA curves for 16%, 50% and 84% IDA of IDR | Sa illustrates that RC Frame-walls crosses the IO performance level and are well below the CP level for all cases which shows acceptable performance.

Keywords: Performance Evaluation, Seismic Responses, Frame-Wall Structures, IS 16700: 2017, NLTHA, IDA.

1. Introduction

A wall-frame structure resists horizontal loading by a combination of shear walls and rigid frames. This system is one of the most prevalent for resisting lateral loads in medium to high rise structures. The height of structure and relative stiffness of walls and frames governs the potential advantages of the structure.

Beyond fifteen stories, the frame structures tend to be uneconomical as they depend predominantly on the rigidity of member connections for their resistance to lateral forces. In order to improve the rigidity and economy, shear walls are introduced in taller buildings. The term frame-shear wall structure is used to represent combination of frames and shear walls.

The method of distributing lateral loads is based on the true interaction behaviour of the frame-shear wall system. Independent deformation shapes of the shear wall and frame, under lateral loads are different as shown in Fig. 1. When these two are secured together by floor slabs or beams, the cumulative pattern of deformation is different. But the interacting forces vary in magnitude and direction along the height of the structure as shown in Fig. 2.

The frame deflects in a shear mode whereas the shear wall predominantly responds by bending as a cantilever. Compatibility of lateral deflection generates interaction between the two. The linear sway of the moment frame, combined with the parabolic sway of the shear wall, results in 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 frame consisting of closely spaced columns and deep beams tends to behave more like a shear wall responding predominantly in a bending mode. And similarly, a shear wall weakened by large openings acts more like a frame by deflecting in a shear mode. The combined structural action, therefore, depends on the relative rigidity of the two, and their modes of deformation. [11], [12]

The behaviour of structures under the effect of seismic ground motions has always been a subject of study. In order to accurately capture the nonlinear seismic response of a

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Proceedings of the 12th Structural Engineering Convention (SEC 2022), NCDMM, MNIT Jaipur, India| 19-22 December, 2022

Published online: December 19, 2022
doi:10.38208/acp.v1.711

structure, complicated dynamic analysis methods and complex material models are required.

Amongst various analysis methods, Nonlinear Time History Analysis (NLTHA) is one of the most accurate method used to compute seismic responses of structures subjected to ground motions. In order to perform NLTHA, properly selected ground motions are applied to the model.

This method of analysis was adopted by the Federal Emergency Management Agency (FEMA) and is considered as the state-of-the-art method to estimate the structural responses under seismic loadings. Incremental Dynamic Analysis (IDA) is an extension of NLTHA. A properly defined 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 called IDA Curve.

Here, analysis and design of RC Frame-Wall Structure with 18, 22 and 26 Storey are carried out as per IS code provisions [6], [7], [8]. The frames are considered to be resting on hard soil and lying in seismic Zone IV. NLTHA as well as IDA with monotonic scaling till numerical non-convergence of above frames are carried out using SeismoStruct software for set of 11 recorded ground motions of past Indian earthquake. Results are then plotted in terms of Interstorey Drift Ratio (IDR) profile, IDA curves and summarized IDA curves, considering performance criteria as per FEMA 356 [4]. Summarized IDA curves are obtained using cross-sectional fractiles at 16%, 50% and 84% records. From the summarized IDA, collapse margin ratios (CMR) are also computed.

As tall buildings are wind sensitive, the basic wind speed considered are as per Indian Standard and wind loads are computed using Gust Factor Method [9].

Incremental Dynamic Analysis (IDA)

IDA curve represents the structural responses and shows structural behaviour subjected to ground motions. IDA cures depend on building stiffness, strength, and ductility to resist seismic loads. In addition, researchers choosing different IM and DM values based on their research objectives, will result in different IDA curves.

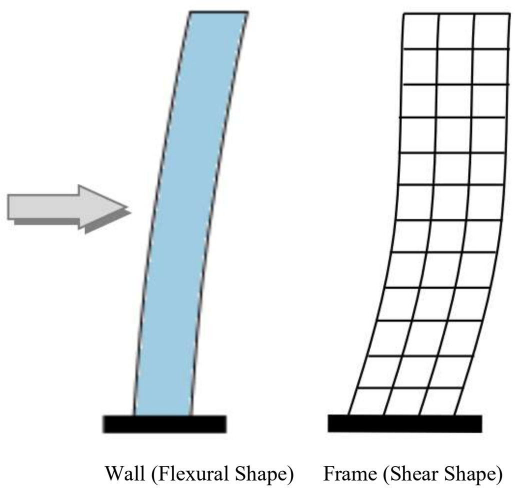

Fig. 1. Deflected Shape of Shear Wall and Frame subjected to Lateral Load

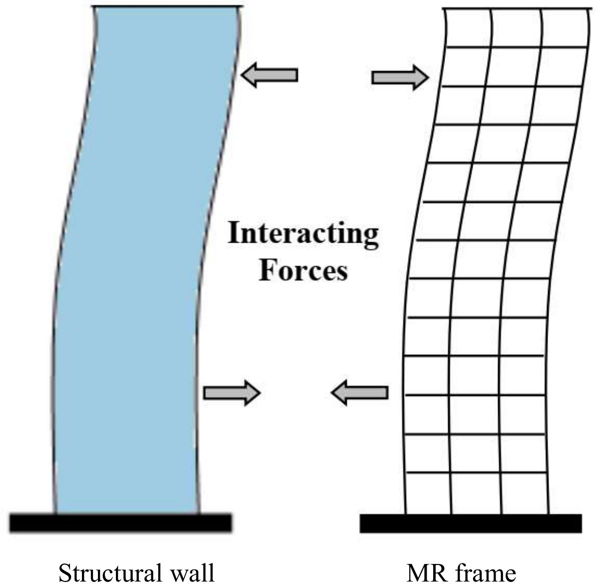

Fig. 2. Shear Wall-Frame Interaction

Slope of the IDA curve gauges the behaviour of the structure. When the slope is linear, the structure has elastic response which means that the proportion of the DM is linear with the IM. Generally, when the scale factor is low, IDA curve is a straight line while, when the scale factor becomes higher, the curve starts to arc indicating nonlinearity. Building is considered as collapse when curve become flat line [13]. Curve softening means building collapses at smaller value of intensity measure and it has larger maximum inter-story drift. Seismic capacity of building model is indicated by the IM values at collapse and different damage values.

2. Structural Analysis and Design

Here, modelling, analysis and design of RC moment resisting frame-wall buildings are done using ETABS 2016 software as per provisions of IS codes. Geometrical configuration of RC moment resisting frame with shear wall is considered as per IS 16700:2017 [6]. The maximum plan aspect ratio (L/B) of the overall building does not exceed 5.0. Also, the maximum building height does not exceed 100 m for structural system of structural wall + moment frame as per criteria of structures located in Zone IV. The maximum value of the slenderness ratio of height (H) to minimum base width (B) does not exceed 8 for structural system of structural wall + moment frame.

Concrete grade of M40 with reinforcement steel of Fe500 grade are used for design. Seismic input includes response reduction factor of 5 with importance factor of 1.2 with structure lying on hard soil. Typical storey height is taken as 3.40 m. Dead load assumed, includes self-weight of beam-columns, wall and slabs and the imposed load considered is 4.00 kN/m^2. Effective moment of inertia for the design of beam and column/shear wall considered is 0.35 I_{g0} and 0.70 I_{g0} respectively, while that for drift for beam and column/shear wall considered are 0.70 I_{g0} and 0.90 I_{g0} respectively. Plan dimension is of 18 m x 36 m as shown in Fig. 3.

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, which is estimated based on realistic section properties shall be limited to H/500. For a single storey the drift limit may be relaxed to hi/400. For earthquake load combinations the drift shall be limited to hi/250.

It is checked that responses obtained through seismic and wind forces are within allowable limit. Table 1 shows linear seismic and wind responses for 22 storey building for illustrative purpose. It indicates that seismic responses are higher compared to wind responses. Hence, analysis and design is carried out considering seismic load combinations.

The design base shear, fundamental time period, spectral acceleration at DBE (Design Basis Earthquake) and modal time period are shown in Table 2.

2.1 Design

RC Moment Resisting Frame-Wall Structure with 18, 22 and 26 storey are designed distinctly for load combinations as per Indian Standards. Cross section dimensions and design reinforcement of columns, beams and shear walls are designed for envelope of load combinations.

Dimensions of beam is kept such that steel reinforcement computed for hogging and sagging moments range from maximum 1.10% steel 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 with boundary elements are designed with maximum 2.54% of steel reinforcement. Footing design of buildings is done using SAFE 2016 software as per provisions of IS code. Buildings are considered to be resting on hard soil with safe bearing capacity (SBC) of soil as 350 kN/m^2.

Table-1. Seismic and Wind Responses

Responses	Seismic Analysis	Wind Analysis
Base Shear (kN)	3897.34	3252.42
Base Overturning Moment (kN.m)	162512.49	159370.73
Top Displacement (mm)	155.05	96.49 < 149.60
Interstorey Drift Ratio (%)	0.2658 < 0.40	0.1544 < 0.25

Table-2. Design Base Shear and Time Period

Storey	First Mode Time Period (s)	Fundamental Time Period (s)	DBE, S_a (g)	Base Shear, V_{bx} (kN)
18	4.26	1.298	0.092	3847.24
22	4.64	1.587	0.076	3897.34
26	5.49	1.875	0.064	4113.04

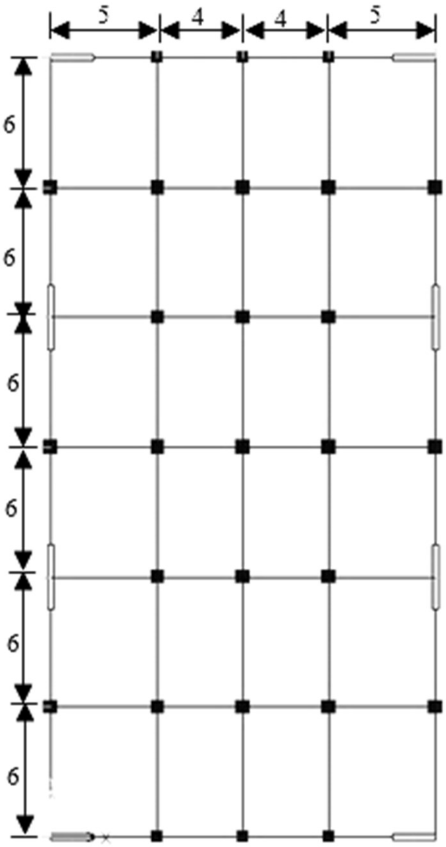

Fig. 3. Typical Plan Layout of Wall-Frame Structure

2.2 Nonlinear Modelling

Nonlinear analysis was done in SeismoStruct software [10] with the uniaxial nonlinear confinement concrete model. This concrete model was programmed by Madas (1993) using both constitutive relationship and cyclic rules proposed by Mandar et al (1988) and Martinez-Rueda and Elnashai (1997), respectively. Also, reinforcement steel used is a uniaxial bilinear stress strain model. This simple model is characterized by easily identifiable calibrating parameters and by its computational efficiency.

Inelastic Force-Based Plastic Hinge Frame type element featuring distributed inelasticity and forced based formulation are used. It concentrates such inelasticity within a fixed length of the element. The advantages of such formulation are not only a reduced analysis time, but also a full control of the spread of inelasticity. The number of section fibers used in equilibrium computations carried out at the element's end sections 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, cantilevered column P-delta effect is considered for RC columns and shear walls. Fixed support is assumed for the foundations. Top of the building is free to move both translationally and rotationally.

2.3 Past-Earthquake Ground Motions

Available records from database that met the following criteria were selected: [2], [3]

Ground motions having a magnitude greater than 5.5. The site condition classified as rock. The recording is made in the far field which has source-to-site distance greater than 10 km. To avoid potential event-based bias in record sets, maximum two records are selected per earthquake. Peak Ground Acceleration (PGA) are more than 0.10g and Peak Ground Velocity (PGV) are more than 10 cm/s.

Selected earthquake data were recorded in different locations pan India. Earthquake names, years of occurrence, magnitude, site source distance, PGA and PGV are shown in Table 3.

Response spectrum of 11 recorded ground motions of past Indian earthquake along with design response spectrum are shown in Fig. 4.

3. Results

NLTHA and IDA are carried out using SeismoStruct software to evaluate structural seismic performance by applying set of 11 recorded ground motions of past Indian earthquake to the structures for parametric study. In order to cover the entire range of the structural response from elasticity to global dynamic instability, each of the selected ground motions were scaled up or down several times and applied to the structure.

Table-3. Ground Motion Properties

Sr. no.	Earthquake Name	Year	Magnitude (M)	PGA (g)	PGV (cm/s)
1	Chamoli	1999	6.6	0.36	45
2	Uttarkashi	1991	7.0	0.25	30
3	Uttarkashi	1991	7.0	0.31	20
4	IB88	1988	7.2	0.34	23
5	IB88	1988	7.2	0.34	21
6	India-Burma	1997	5.6	0.16	24
7	India-Burma	1997	5.6	0.15	21
8	Bhuj	2001	7.6	0.11	11
9	Sikkim	2011	6.9	0.34	-
10	Sikkim	2011	6.9	0.29	-
11	Gorkha	2015	7.8	0.26	30

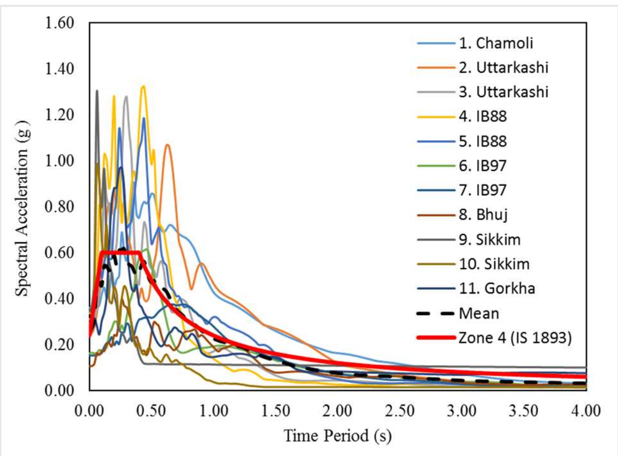

Fig. 4. Ground Motion Response Spectra and Design Response Spectrum

3.1 Interstorey Drift Ratio

Interstorey Drift Ratio (IDR) is defined as the ratio of relative horizontal displacement of two adjacent floors and corresponding storey height. It is one of the most important design parameters in the seismic design codes. The IDR profile obtained by nonlinear time history analysis for 11 recorded ground motions of past Indian earthquake for 18, 22 and 26 storey buildings are shown in Fig. 5, Fig. 6 and Fig. 7 respectively.

Median value of IDR is also plotted along with individual profiles. It is observed from the results of NLTHA, that maximum IDR does not exceed target drift limit of 2%. Hence, the frames show satisfactory performance under seismic loading.

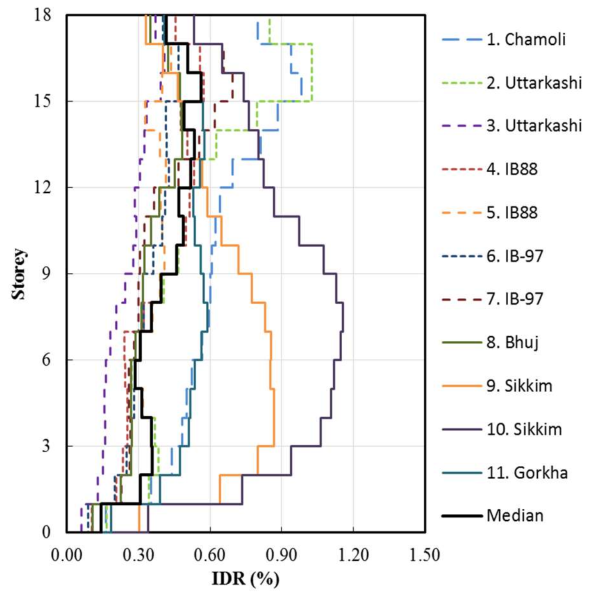

Fig. 5. IDR Profile for 18 Storey Building

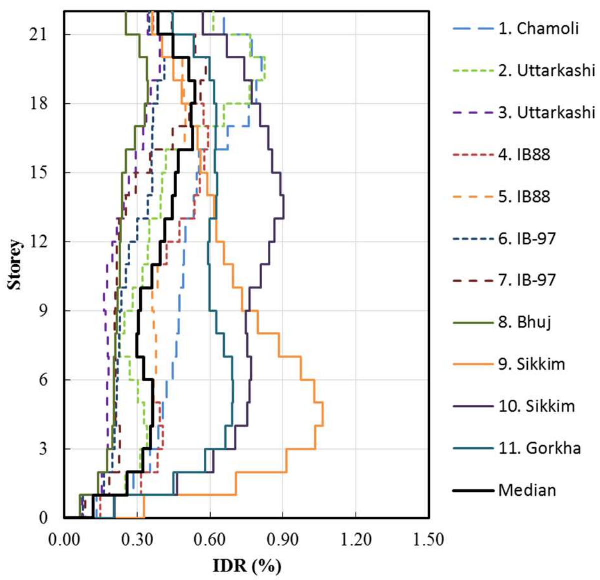

Fig. 6. IDR Profile for 22 Storey Building

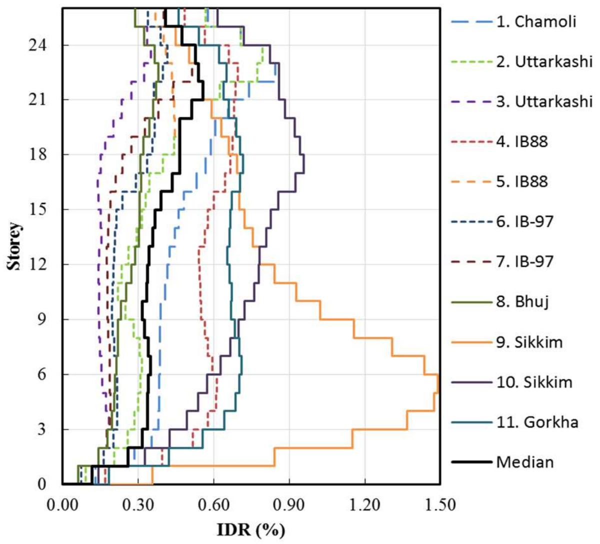

Fig. 7. IDR Profile for 26 Storey Building

3.2 Plotting IDA Curves

IDA curve plotting is a three step procedure as discussed below after selection of ground motions: [1]

Selection of Intensity Measure (IM):

Taken as 5% Damped Spectral Acceleration. It is more efficient than PGA, because Sa is structure specific as compared to PGA which is site specific. Therefore, Sa is taken as the choice for IM.

Selection of Damage Measure (DM):

Selected as Maximum Interstorey Drift Ratio. It is well related to estimating structure's damage state.

IDA Curve generation by interpolation:

After selecting the IM and DM entities, their values are computed after each dynamic analyses for scaled ground motions. Set of more discrete points for each ground motion record are obtained which resides in the IM-DM plane, lying on its IDA curve. All these points are then connected using spline interpolation which gives a realistic representation of IDA curve. Having the complete curve available, it is now possible to calculate DM values at any arbitrary levels of IM.

The results of IDA Curves for 18, 22 and 26 storey buildings are shown from Fig. 8 to Fig. 10.

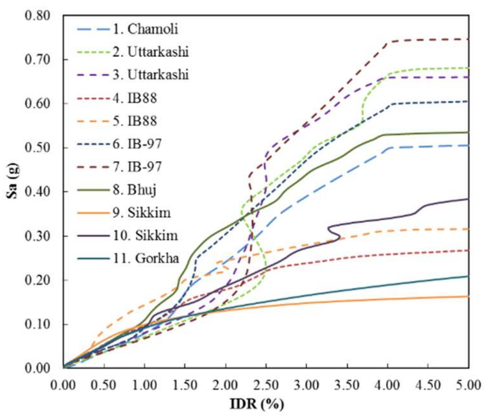

Fig. 8. Individual IDA Curves for 18 Storey Building

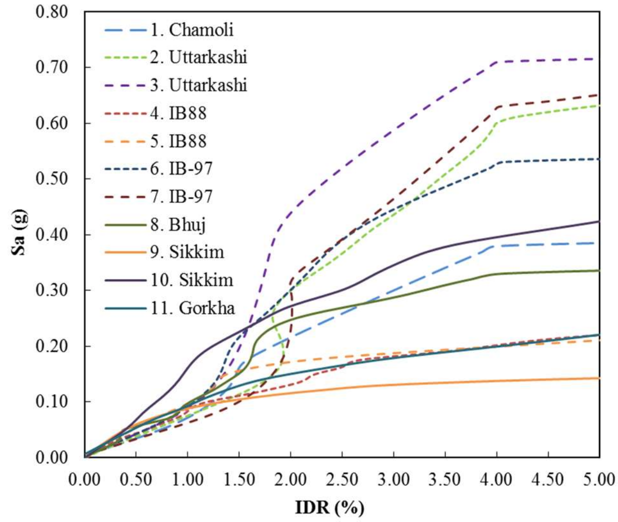

Fig. 9. Individual IDA Curves for 22 Storey Building

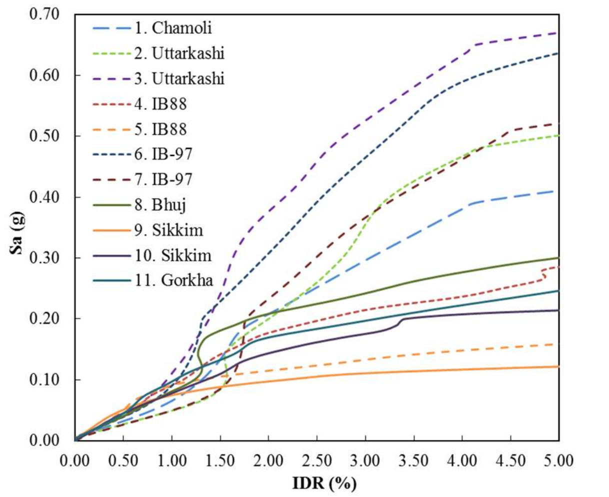

Fig. 10. Individual IDA Curves for 26 Storey Building

3.3 Summarizing IDA Curves

It becomes essential to summarize the data as large amount of record to record variability can be seen in the IDA curve. Various methods are available for appropriate summarization and to quantify the randomness. It is necessary to employ appropriate summarization techniques that will reduce this data distribution of DM given IM and to the probability of exceeding any particular limit-state at the given IM level. Threshold IDR (%) for IO, LS and CP limit-states are 0.50, 1.00 and 2.00 respectively. Global Dynamic Instability (GI) occurs when any increase in the IM results in practically infinite DM response resulting in the flat line.

The limit-state capacities can be summarized into central value (e.g. mean or median) and a measure of dispersion (like the standard deviation, or the difference between two fractiles). Among the several methods available to summarize the IDA curves, the cross-sectional fractiles are perhaps the most flexible and robust with respect to the infinite DMs introduced by the flat lines. Thus, it is chosen to calculate the 16%, 50% and 84% fractile values of DM and IM capacity for each limit-state.

The summarized capacities for each limit state for all buildings are given in Table 4.

Table-4. Summarized Capacity for Each Limit State

Storey	Performance Levels	Sa (g)			MCE (Sa), g	CMR
		16%	50%	84%		
18	IO	0.058	0.050	0.037	-	-
	LS	0.102	0.092	0.075	-	-
	CP	0.265	0.183	0.140	-	-
	GI	0.670	0.500	0.260	0.185	2.70
22	IO	0.057	0.044	0.037	-	-
	LS	0.097	0.090	0.075	-	-
	CP	0.303	0.230	0.144	-	-
	GI	0.630	0.390	0.200	0.151	2.58
26	IO	0.047	0.040	0.030	-	-
	LS	0.096	0.080	0.057	-	-
	CP	0.260	0.200	0.123	-	-
	GI	0.590	0.280	0.210	0.128	2.19

The ratio between median collapse intensity (S_{CT}) and ground motion intensity (S_{MT}) at Maximum Considered Earthquake (MCE) is defined as collapse margin ratio (CMR) [5], which is the primary parameter used to characterize the collapse safety of the structure. The median collapse intensity can be obtained by increasing the intensity until just over one-half of the records cause collapse. The lowest intensity at which one-half of the records causes collapse is the median collapse intensity. The MCE intensity is obtained from the response spectrum of MCE ground motions at the fundamental period, T.

$$CMR = \frac{S_{CT}}{S_{MT}}$$

Under the suitable assumption of continuity and monotonicity of the IDA curves, the fractiles can be interpreted. For illustration purpose, fractile for 22 storey building is explained as: It is observed from summarized curve that in order to generate threshold demand IDR of 1% which is Life Safety Performance Level, 84% of the records can reach or exceed the IM levels (Sa) of 0.075 g, 50% of the records can reach or exceed the IM levels (Sa) of 0.090 g, and 16% of the records can reach or exceed the IM levels (Sa) of 0.097 g. Likewise, for Collapse Prevention Performance Level, 84% of records with Sa ≥ 0.144 g, 50% of records with Sa ≥ 0.230 g and 16% of records with Sa ≥ 0.303 g has capacity to reach threshold IDR of 2%. The collapse margin ratio (CMR) varies from 2.19 to 2.70 which shows safety to the global collapse of structure. Summarized IDA plots are shown in Fig. 11 to 13.

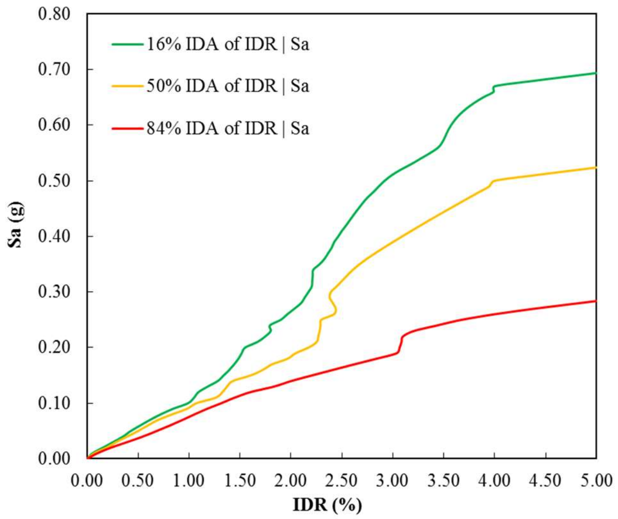

Fig. 11. Summarized IDA Curves for 18 Storey Building

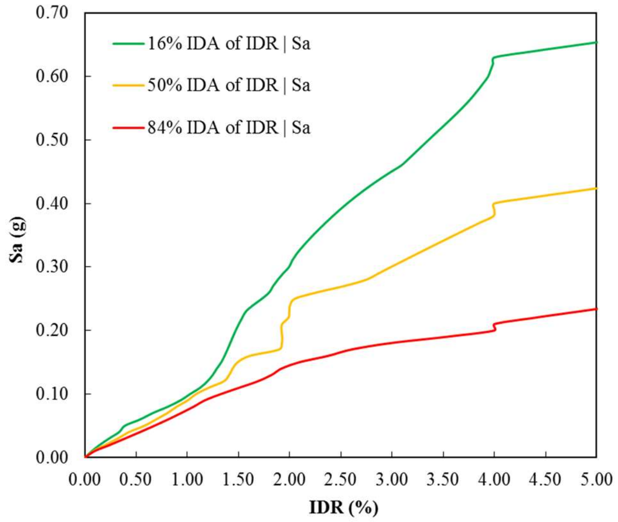

Fig. 12. Summarized IDA Curves for 22 Storey Building

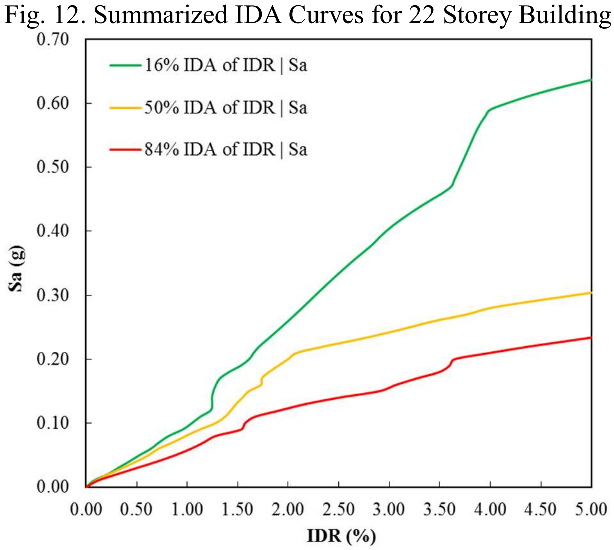

Fig. 13. Summarized IDA Curves for 26 Storey Building

4. Conclusion

In this study, RC moment resisting frame-wall structure with 18, 22 and 26 storeys are analyzed for seismic zone IV and designed as per IS code provisions, considering both gravity and seismic loads. The performance evaluation of above frames is done using NLTHA and IDA using SeismoStruct software for set of 11 recorded ground motions of past Indian earthquake.

The following conclusions have been drawn from this study:

It is observed from the results of NLTHA, that maximum Interstorey Drift Ratio does not exceed target drift limit 2% for all frames. Hence, the frames show satisfactory performance under seismic loading.

From individual IDA curves plotted, it is seen that buildings show different behavior subjected to each different ground motion records.

It is observed from results of summarized IDA curves that the median collapse capacity of buildings reduces with increase in number of storeys or increase in height of building.

IDA curves shows that the RC frame-walls designed for Design Basis Earthquake (DBE) crosses the Immediate Occupancy Performance Level but are well below the Collapse Prevention Level for all cases which indicates satisfactory performance.

Collapse margin ratio (CMR) varies from 2.70, 2.58 to 2.19 for 18, 22 and 26 storeys respectively which shows overall safety against global collapse for all considered structures. But it is observed that, with increase in number of storey, CMR ratio reduces and hence, safety to the global collapse of structure reduces with increase in height of structure.

Disclosures

Free Access to this article is sponsored by SARL ALPHA CRISTO INDUSTRIAL.

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