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EFFECT OF ACCIDENTAL ECCENTRICITY ON COLLAPSE PROBABILITIES OF MID-RISE RC FRAME BUILDINGS

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ABSTRACT

The accidental eccentricity provisions of IS 1893 are compared with other national codes and examined using Incremental Dynamic Analysis procedure in order to estimate collapse probabilities of mid-rise (4-, 8- and 12-storied) RC frame buildings. The buildings are designed for regular symmetric configuration following Indian codes and are considered with varying accidental eccentricities of 0% (regular symmetric), 5%, 10%, and 15%. The accidental eccentricity is introduced by changing the mass distribution on floor plan, keeping the total mass at each floor same as for a corresponding regular symmetric building. The obtained results (dynamic characteristics, torsional irregularity indices, dynamic capacity curves, collapse margin ratio and collapse probabilities) for regular symmetric buildings and buildings with accidental eccentricity are compared. It is observed that buildings with accidental eccentricity, on average, exhibit 1.8 and 1.4 times higher probability of collapse as compared to the corresponding regular symmetric buildings, for DBE and MCE demands, respectively.

KEYWORDS: Accidental Eccentricity; Collapse Probabilities; Fragility Analysis; Incremental Dynamic Analysis; RC Frame; Torsion

INTRODUCTION

The torsional response of buildings under seismic excitation is not only caused by the irregular configuration resulting in non-symmetrical distribution of mass and stiffness, but also due to other unforeseen factors such as incoherent ground-motion characteristics, change of the structure’s occupancy, difference between calculated and actual centres of mass and stiffness, as well as mass and stiffness contributions due to the presence of non-structural components which are generally ignored at the design stage of the building. All of these unforeseen effects lead to some torsional response even for a perfectly symmetric building. In order to cater to these effects, modern seismic design codes (IS 1893 Part 1, 2002; IS 1893 Part 1, 2016; ASCE 7-10, 2010; EN 1998, 2005; NZS 1170.5, 2004) introduced the concept of accidental eccentricity. The accidental eccentricity plays an important role in the seismic design of buildings, though being ignored by designers most of the times, because of the complexity in its implementation, as it requires additional analysis efforts with the centre of mass displaced from its original position in the direction perpendicular to the excitation by a predefined eccentricity value.

Many studies have focused on the adequacy of the code provisions on accidental eccentricity and its effects on the structure’s torsional behaviour for both elastic and inelastic single-story buildings (e.g., Chopra and Goel, 1991 and Stathopoulos and Anagnostopoulos, 2003) as well as multi-story elastic and inelastic buildings with symmetric (e.g., De la Llera and Chopra, 1994 (a, b, c, d); De la Llera and Chopra, 1995 and Stathopoulos and Anagnostopoulos, 2005) and plan asymmetric structures (e.g., Chandler and Duan, 1991 and De Stefano et al., 1993). It has been reported that the uncertainties in the location of the centres of mass and stiffness account for over 70% of the total increase in response due to accidental torsion (De la Llera and Chopra, 1994b; De la Llera and Chopra, 1994c).

An important issue while studying the effect of accidental torsion (eccentricity) are the disagreements among various researchers about whether the accidental eccentricity should be considered in the design of the models in order to study the effect of accidental torsion. For example, Chandler and Duan (1991); Chopra and Goel (1991) and De Stefano et al. (1993) suggested that the building models used to study the effect of accidental torsion should not be designed for accidental eccentricity. Contrary to this, Rutenberg et al. (1992); Wong and Tso (1994) and Correnza et al. (1995) considered accidental eccentricity as a design parameter for their study. Findings of more recent studies conducted by Stathopoulos and

Anagnostopoulos (2003, 2005) suggested that the inelastic response of multi-story buildings is very little influenced when accidental eccentricity is considered in their design.

To the best of the authors' knowledge, none of the previous studies has focused on the effect of accidental eccentricity on collapse probabilities of code-designed buildings. In the present study, an attempt is made to investigate the effect of accidental eccentricity on collapse risk of mid-rise RC frame buildings. The investigation includes both regular symmetric buildings (i.e., buildings with coincident centre of mass and centre of rigidity) as well as buildings with accidental eccentricity (i.e., buildings with non-coincident centres of mass and centre of rigidity). The investigation of the building models is carried out using the Incremental Dynamic Analysis (IDA) method. The building models are designed according to the relevant Indian codes for regular symmetric case only and thereafter accidental eccentricity is introduced by changing the distribution of mass on the floors. A suite of 30 recorded earthquake ground-motions is used in order to derive the buildings' dynamic capacity curves. Collapse margin ratios (FEMA P695, 2009) and fragility functions are examined to assess the effect of accidental eccentricity on the collapse performance of buildings.

CODE PROVISIONS

The major national seismic design codes differ in the amount of accidental eccentricity to be considered in the analysis and design of buildings and in defining the torsional irregularity. The accidental eccentricity which arises due to the torsional component of the seismic ground motion and due to the unknown discrepancies in the building (as defined in the Introduction section) is expressed as a percentage of the plan dimension orthogonal to the direction of the seismic action. Some of the seismic design codes, e.g. Indian code IS 1893 Part 1 (2002); IS 1893 Part 1 (2016); ASCE 7-10 (2010) and EN 1998 (CEN, 2005), consider 5% of the plan dimension perpendicular to the direction of the seismic action as the accidental eccentricity. On the other hand, New Zealand code (NZS 1170.5, 2004) considers 10% of the plan dimension perpendicular to the direction of the seismic action as accidental eccentricity. NZS 1170.5 also states that when the seismic actions are not applied in a direction parallel to the principal orthogonal axes of the structure, then the accidental eccentricity is assumed to lie on the outline of an ellipse. The semi-axes of the ellipse, along the principal axes of the structure, are equal to the accidental eccentricity of 10%.

The accidental eccentricity leads to a torsional seismic response of the building. Different codes of the world quantify this torsional response using different parameters. IS 1893 (2002) and ASCE 7 (2010) define the torsional irregularity in terms of the ratio of the maximum elastic inter-storey drift to the average inter-storey drift of the floor diaphragm. Recently, the Indian seismic design code has been revised, and in the revised code (IS 1893 Part 1, 2016), the torsional irregularity is defined in terms of the ratio of the maximum elastic displacement to the minimum displacement of the floor diaphragm. NZS defines the torsional irregularity in terms of the ratio of the maximum elastic edge displacement to the average edge displacement of the floor. The European code EN 1998 defines the torsional irregularity in terms of the ratio of eccentricity e (i.e., distance between centre of mass and centre of rigidity) to torsional radius r (i.e., squareroot of the ratio of the torsional stiffness to the translational stiffness). The provisions of different codes are summarized in Table 1.

All the considered codes recommend that irregular buildings should be analysed using 3D models. NZS 1170.5 also states that the accidental torsional effects can be considered either by shifting the centre of mass (without modifying the rotational inertia of the floor about the nominal centre of mass) or by considering the line of action of earthquake loading at a distance equal to the accidental eccentricity from the nominal centre of mass. In order to take into account the torsional effects in the design of torsionally irregular buildings, ASCE 7 recommends an amplification of forces (which are obtained by considering the accidental eccentricity) by a so-called torsional amplification factor B_i:

$$B_i = \left(\frac{d_{max}}{1.2\ d_{avg}}\right)^2 \tag{1}$$

where, B_i is the torsional amplification factor with a lower-bound value of unity and upper-bound value of 3, d_{max} is the maximum inter-storey drift of the floor under consideration, and d_{avg} is the average inter-storey drift of the floor under consideration.

It can be seen that different codes differ in the magnitude of the accidental eccentricity, the definition of torsional irregularity and the design measures adopted for the seismic design of torsionally irregular buildings. In the present study, buildings are designed following Indian standards, and the definitions of accidental eccentricity and torsional irregularity are considered as per both, the older (IS 1893 Part 1, 2002) and the revised (IS 1893 Part 1, 2016) version.

Table 1: Comparison of Provisions for Accidental Eccentricity and Torsional Irregularity as per Different Codes

Code	Accidental Eccentricity [1]	Torsional Irregularity Criteria		
		d_{max}/d_{avg}	d_{max}/d_{min}	e/r
IS 1893 Part 1 (2016)	0.05 b	-	1.50	-
IS 1893 Part 1 (2002)	0.05 b	1.20	-	-
IS 1893 (Draft) (2005)	0.05 b	1.20, 1.40 [2]	-	-
ASCE 7-10 (2010)	0.05 b	1.20, 1.40 [2]	-	-
EN 1998 (CEN, 2005)	0.05 b	-	-	0.30
NZS 1170.5 (2004)	0.10 b	1.40	-	-

[1] b is the plan dimension orthogonal to the direction of the seismic action

[2] value represents extreme torsional irregularity

NUMERICAL STUDY

In the present study, a set of symmetric (regularity in plan) and unidirectionally mass-eccentric reinforced-concrete Special Moment Resisting Frame (SMRF) buildings is considered. The buildings are modelled with 4, 8 and 12 storeys, thereby representing the class of mid-rise buildings in India. All buildings have constant storey heights equal to 3.3 m and identical plan layouts as shown in Figure 1. The plan shape was decided based on a field survey in the National Capital Region around New Delhi (DEQ, 2009; Haldar, 2013). The mass-eccentric building configurations are generated by varying the distribution of mass linearly along the X-direction in such a way that the total mass at the considered floor level remains the same as that of the corresponding regular symmetric building and the mass-eccentric building configurations result in an eccentricity equal to the targeted accidental eccentricity viz. 5% , 10% and 15%.

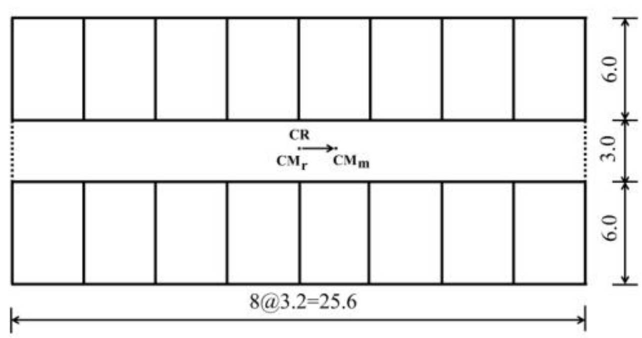

Fig. 1 Plan of the considered building models (CR - centre of rigidity, CM_r- centre of mass of regular building, and CM_m - centre of mass of mass-eccentric building, longer and shorter dimensions are considered along x and y directions, respectively).

Modelling and Analysis

The building models are created in the proprietary finite element code SAP 2000 v 14.2.4 (CSI, 2010). Beams and columns are modelled using 3-D frame elements while slabs are defined as rigid diaphragms. The cracked section properties of beams and columns are derived following ASCE 41-06 (2007). Dead loads and live loads on the buildings are assigned according to IS 875 Part 1 (1987) and IS 875 Part 2 (1987), respectively. The buildings are designed as Special Moment Resisting Frame (SMRF) buildings following the Indian standard IS 1893 Part 1 (2002), for seismic zone V on soil type I (i.e., hard soil/rock). P-delta effects are considered in the analysis as well as in the design.

The results obtained from both the modal analysis and linear dynamic analysis are presented in Table 2. The dynamic characteristics of mass-eccentric building models suggest that on average 4%, 9% and 11% of the total mass in the transverse direction of the building is participating with the torsional (rotational) mode of vibration for the 5%, 10% and 15% mass-eccentric buildings, respectively. The results obtained from the linear dynamic analysis show that all mass-eccentric building models have torsional irregularity indices d_{max}/d_{avg} and d_{max}/d_{min} greater than 1.2 and 1.5, respectively (Table 2).

Table 2: Dynamic Characteristics and Torsional Irregularity Indices

Building Model	Building Height	Mode Number	T (s)	α_{mx}	α_{my}	e/r	d_{max}/d_{avg}	d_{max}/d_{min}
Regular symmetric building	4-storeys	Mode 1	1.75	0.00	0.82	0.00	1.00	1.00
		Mode 2	1.39	0.00	0.00			
		Mode 3	1.22	0.85	0.00			
	8-storeys	Mode 1	2.90	0.00	0.80	0.00	1.00	1.00
		Mode 2	2.43	0.00	0.00			
		Mode 3	2.20	0.81	0.00			
	12-storeys	Mode 1	3.38	0.00	0.81	0.00	1.00	1.00
		Mode 2	2.75	0.00	0.00			
		Mode 3	2.45	0.83	0.00			
5% Mass-eccentric buildings	4-storeys	Mode 1	1.77	0.00	0.78	0.11	**1.23**	**1.68**
		Mode 2	1.36	0.00	0.03			
		Mode 3	1.22	0.85	0.00			
	8-storeys	Mode 1	2.95	0.00	0.75	0.12	**1.26**	**1.75**
		Mode 2	2.38	0.00	0.05			
		Mode 3	2.20	0.81	0.00			
	12-storeys	Mode 1	3.44	0.00	0.78	0.11	**1.23**	**1.69**
		Mode 2	2.72	0.00	0.03			
		Mode 3	2.45	0.83	0.00			
10% Mass-eccentric buildings	4-storeys	Mode 1	1.83	0.00	0.74	0.22	**1.34**	**2.16**
		Mode 2	1.29	0.00	0.07			
		Mode 3	1.22	0.85	0.00			
	8-storeys	Mode 1	3.07	0.00	0.70	0.24	**1.33**	**2.08**
		Mode 2	2.24	0.00	0.11			
		Mode 3	2.20	0.81	0.00			
	12-storeys	Mode 1	3.54	0.00	0.73	0.22	**1.33**	**2.10**
		Mode 2	2.59	0.00	0.08			
		Mode 3	2.45	0.83	0.00			
15% Mass-eccentric buildings	4-storeys	Mode 1	1.90	0.00	0.72	**0.33**	**1.39**	**2.44**
		Mode 2	1.22	0.85	0.00			
		Mode 3	1.19	0.00	0.09			
	8-storeys	Mode 1	3.20	0.00	0.68	**0.36**	**1.37**	**2.22**
		Mode 2	2.20	0.81	0.00			
		Mode 3	2.08	0.00	0.12			
	12-storeys	Mode 1	3.68	0.00	0.71	**0.33**	**1.37**	**2.30**
		Mode 2	2.45	0.83	0.00			
		Mode 3	2.42	0.00	0.11			

T – Period of vibration; e – eccentricity; r – torsional radius; α_{mx} and α_{my} represent the modal mass participation ratio in the x and y direction, respectively. The values of irregularity indices shown in bold face indicate torsional irregularity.

It is interesting to note that both the definitions of torsional irregularity (IS 1893 Part 1, 2002 and ASCE 7, 2010 definition based on d_{max}/d_{avg} as well as IS 1893 Part 1, 2016 definition based on

d_{max}/d_{min}) lead to a classification of these mass-eccentric buildings as 'torsionally irregular' buildings. Contrary to this, these buildings are classified as 'torsionally regular' as per NZS 1170.5; as the obtained ratio of the maximum edge displacement to the average edge displacement of the floor is less than 1.40 for all the considered building models.

The estimation of the torsional radius (r) for multi-storey buildings is a complex and cumbersome task. In the present study, the stiffness of individual frames is obtained by performing pushover analysis using mode proportional distribution of lateral load along the height of the frame. The initial stiffness of individual frames, estimated from the pushover curve, is used to estimate the torsional radius of the building. It can be observed that only mass-eccentric buildings with 15% eccentricity qualify as 'torsionally irregular' according to the definitions of EN 1998 (CEN, 2005). Hence, it can be seen that the buildings which are torsionally irregular according to IS 1893 and ASCE 7 are torsionally regular as per NZS 1170.5 and EN 1998 (except buildings with 15% eccentricity, which are classified as 'torsionally irregular' as per EN 1998). This observation shows that significant variation exists among the seismic design codes about the definition of torsional irregularity. It is interesting to note that the consideration of 5% accidental eccentricity alone results in the building to be classified as a torsionally irregular building, even in the case of a perfectly symmetric building (Table 2). This means that each building (even if it is perfectly symmetric) is to be considered as a torsionally irregular building as per the IS 1893 (2002; 2016) and ASCE 7 (2010). This observation is not specific to the set of buildings considered in this study, as it could be observed in earlier studies as well (e.g., Tezcan and Alhan, 2003).

Incremental Dynamic Analysis

The seismic response of an inelastic structure can be most reliably estimated using the non-linear time-history (dynamic) analysis (NLTHA). The NLTHA is performed by incrementally increasing the intensity of the input excitation until the structure finally reaches collapse. The procedure is called Incremental Dynamic Analysis (IDA; Vamvatsikos and Cornell, 2002). IDA is considered a very powerful tool for estimating the seismic capacity and for fragility analysis, as it enables a direct estimation of the record-to-record (inter-event) variability in structural response. However, the application of this methodology requires a careful selection and scaling of ground-motion records in order to obtain results of reasonable accuracy.

Shome et al. (1998) suggested that for mid-rise buildings, 20 records belonging to a bin of large magnitudes (moment magnitudes M_w=6.5-6.9) and moderate distances are usually enough in order to provide sufficient accuracy in the estimation of seismic demands. Maniyar and Khare (2011), Vamvatsikos and Cornell (2002), and Shakib and Pirizadeh (2014) used records with moment magnitudes (M_w) in the range of 5.4-6.9, 6.5-6.9 and 6.0-7.4, respectively, with source-to-site distances (R) varying between 0-90 km. FEMA P695 (2009) also considers that large magnitude events pose the greatest risk of building collapse, and therefore recommends the use of records with magnitudes (M_w) greater than 6.5 for structural collapse capacity assessment. FEMA P695 (2009) defines a threshold distance of 10 km to classify records as near-field or far-field events. It also states that more than two records from the same event should not be used in order to avoid event-related bias. In the present study, 30 records were selected from the PEER database (PEER, 2011) for earthquakes with magnitudes $M_w > 6.5$ and source-to-site distances R>10 km recorded on NEHRP site class B, C and D representing soft rock, stiff soil and soft soil sites. The additional details of these ground-motion records can be found in Surana et al. (2016).

The scaling of seismic ground motion is an important step in order to assess the seismic collapse capacity of the structures. Vamvatsikos and Cornell (2002) suggested that for unidirectional IDA, the most commonly used ground-motion intensity measure (IM) is the 5% damped elastic spectral acceleration at the fundamental mode, i.e., S_a (T_1, 5%) (Vamvatsikos and Cornell, 2002). The reason to choose S_a (T_1, 5%) is that it shows less dispersion in the estimated capacity of fundamental mode-dominated structures. Shakib and Pirizadeh (2014) defined $S_{a,GM}$ (0.5 T_1-1.5 T_1, 5%) as the geometric mean of spectral accelerations over a period range of 0.5 T_1 to 1.5 T_1 at an interval of 0.05 T_1, and used it as the ground motion record scaling parameter for vertically irregular setback structures. The advantage of selecting $S_{a,GM}$ (0.5 T_1-1.5 T_1, 5%) as the scaling parameter is that it considers the effect of higher modes as well as period elongation, in scaling.

The selection of an engineering demand parameter (EDP) to capture the collapse response is dependent on whether the structural or non-structural performance is to be evaluated. The maximum inter-storey drift ratio (θ_{max}) is well correlated with both structural performance and global dynamic instability as it is directly related to a building's side-sway collapse mechanism, while peak floor accelerations are well correlated with non-structural or content damage. The maximum inter-storey drift ratio (θ_{max}) is the most commonly used EDP in literature, e.g. Vamvatsikos and Cornell (2002); Haselton et al. (2011); Afarani and Nicknam (2012); and Shakib and Pirizadeh (2014). In the present study, both regular symmetric as well as asymmetric buildings are considered. Hence, a common set of IM, i.e., S_a $(T_1$, 5%) and EDP (θ_{max}) are used.

In the present study, the non-linearity has been incorporated in the frame elements using lumped-plasticity hinges. The deformation-controlled flexural failure of the beams (uniaxial moment hinges) and columns (interacting P-M-M hinges) has been modelled using an idealized force-deformation curve (backbone curve) as per ASCE 41-06 (2007) (Figure 2). This idealized force-deformation curve conservatively accounts for the cyclic deterioration effects (PEER/ATC 72-1, 2010) and has a post-peak strain softening branch which is the most influential in structural collapse assessment (Haselton et al., 2011). The capacity shear design provisions are included in the design, hence shear failure has not been modelled in this study. The experimental evidence reveals that the SMRF buildings are not subjected to joint shear failure or axial failure prior to side-sway collapse (FEMA P695, 2009). Therefore, these modes of failure are not considered in the present study. Material properties, section dimensions, and non-linear modelling parameters for typical beams and columns are presented in Table 3. In order to perform the non-linear dynamic analysis, 5% Rayleigh damping is considered corresponding to the fundamental mode and the mode which corresponds to a total of 95% of mass participation in the direction under consideration. The collapse has been defined as the point where a slight increase in IM causes a very large increase in EDP.

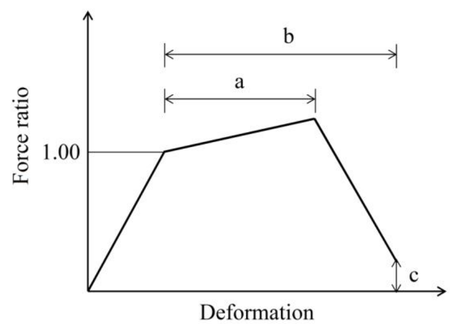

Fig. 2 Force-deformation curve for members under flexure. (Values of modelling parameters a, b, and c are provided in Table 3)

Table 3: Member Dimensions and Modelling Parameters for Typical Beams and Columns

N	Member	Concrete Strength	Rebar Strength	Dimensions	a (rad)	b (rad)	c
4	Beam	M40	Fe500	0.30m x 0.45m	0.025	0.050	0.20
	Column	M40	Fe500	0.35m x 0.35m	0.019	0.029	0.20
8	Beam	M40	Fe500	0.30m x 0.45m	0.025	0.050	0.20
	Column	M40	Fe500	0.35m x 0.35m	0.016	0.025	0.20
12	Beam	M40	Fe500	0.40m x 0.50m	0.025	0.050	0.20
	Column	M40	Fe500	0.40m x 0.40m	0.013	0.021	0.20

N – Number of storeys; and a, b and c are the modelling parameters as defined in Figure 2.

DYNAMIC CAPACITY CURVES

Figures 3-5 present the obtained dynamic capacity curves for the considered regular symmetric and mass-eccentric buildings. The dynamic capacity curves are presented in form of median, 16th percentile and 84th percentile as obtained for the suite of 30 ground-motion records for each building model. The flattening of these curves show collapse of the corresponding building. It can be observed that the median collapse capacity of each mass-eccentric building is lower than the corresponding symmetric case (Figures 3-5).

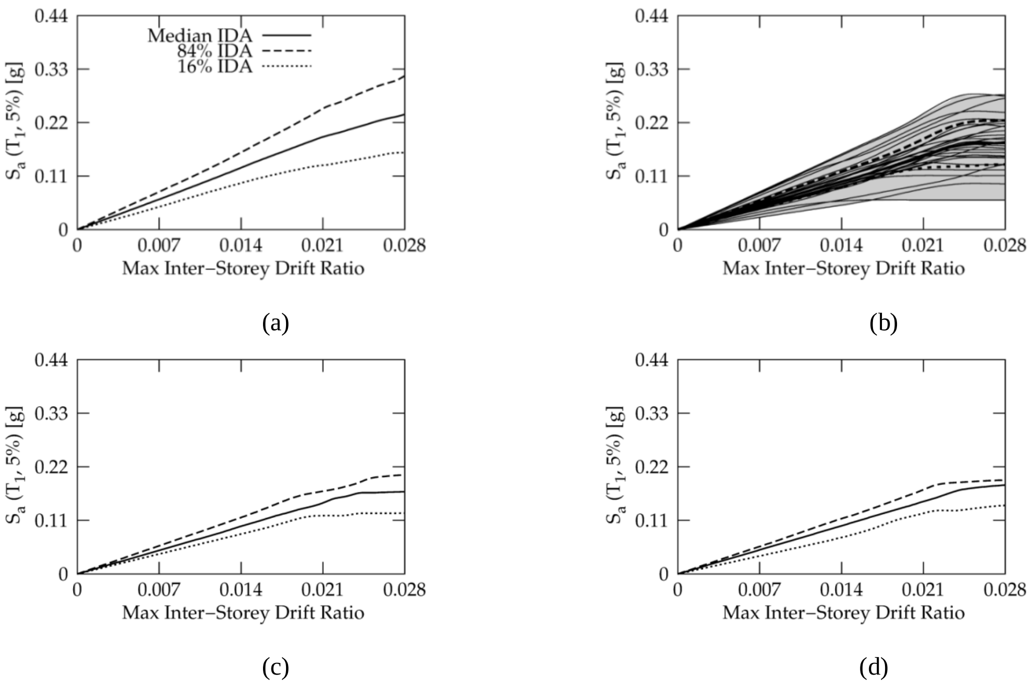

Fig. 3 Dynamic capacity curves for 4-storey buildings: (a) Regular symmetric building, (b) 5% Mass-eccentric building, (c) 10% Mass-eccentric building, and (d) 15% Mass-eccentric building

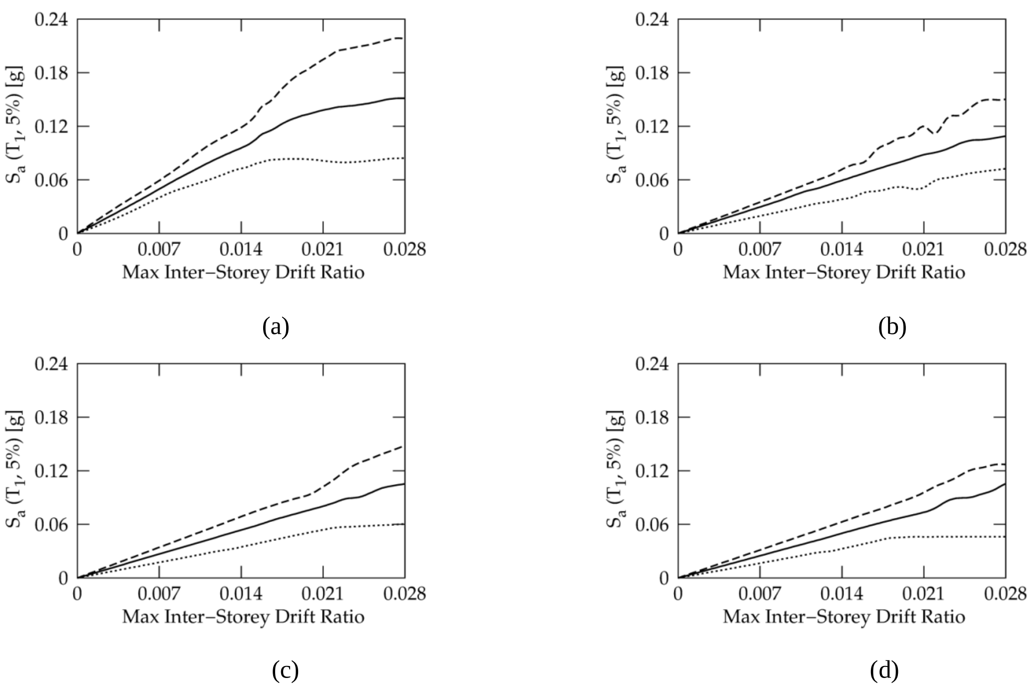

Fig. 4 Dynamic capacity curves for 8-storey buildings: (a) Regular symmetric building, (b) 5% Mass-eccentric building, (c) 10% Mass-eccentric building, and (d) 15% Mass-eccentric building

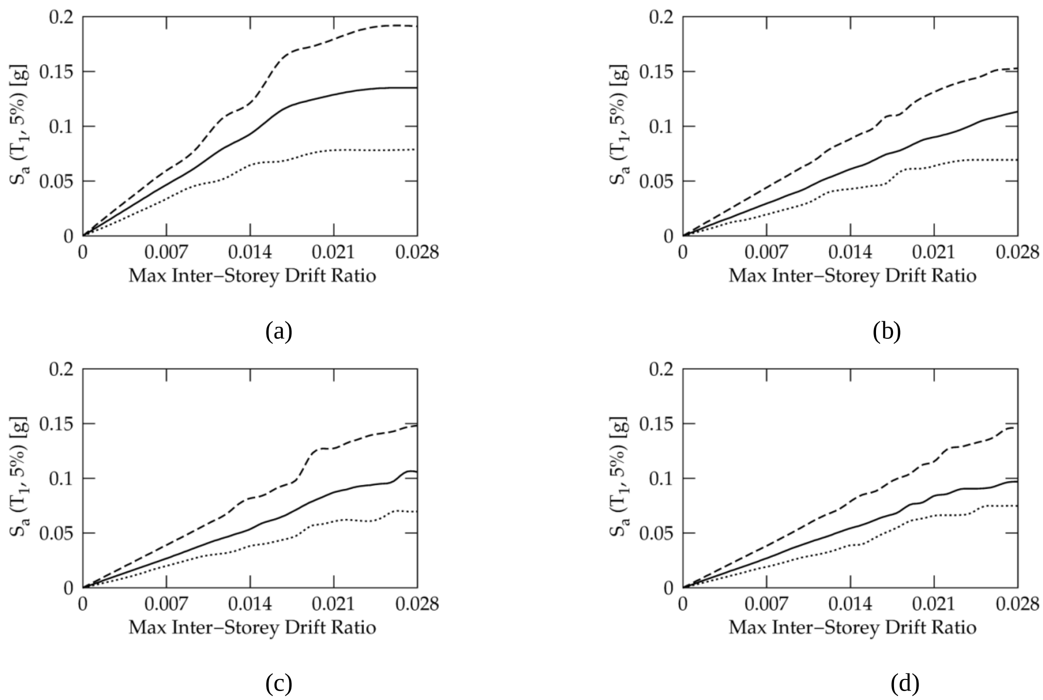

Fig. 5 Dynamic capacity curves for 12-storey buildings: (a) Regular symmetric building, (b) 5% Mass-eccentric building, (c) 10% Mass-eccentric building, and (d) 15% Mass-eccentric building

The damage pattern of the mass-eccentric buildings suggests that the flexible side members yield first, and collapse is governed by failure of beams and columns of the flexible side. It is important to note that the yielding of the flexible side members leads to a shift of the centre of stiffness towards the rigid side leading to a further increase in eccentricity. This additional eccentricity is found out to be more influential than the initial (mass) eccentricity under the inelastic seismic response of buildings (Bugeja et al., 1999).

Table 4 presents the collapse margin ratio (CMR), defined as the ratio of the median collapse capacity $\left(S_{am,Collapse}\right)$ to seismic demand corresponding to the maximum considered earthquake $\left(S_{a,MCE}\right)$ for the considered building models.

Table 4: Collapse Margin Ratio for the Considered Building Models

Building Model	Building Height	$S_{am,\text{ Collapse}}$ (g)	$S_{a,\text{ MCE}}$ (g)	CMR	Reduction in CMR (%)
Regular symmetric building	4-storeys	0.260	0.206	1.26	-
	8-storeys	0.150	0.124	1.21	-
	12-storeys	0.135	0.107	1.26	-
5% Mass-eccentric buildings	4-storeys	0.179	0.203	0.88	30
	8-storeys	0.117	0.122	0.96	21
	12-storeys	0.120	0.105	1.14	09
10% Mass-eccentric buildings	4-storeys	0.167	0.197	0.85	33
	8-storeys	0.112	0.117	0.96	21
	12-storeys	0.113	0.102	1.11	12
15% Mass-eccentric buildings	4-storeys	0.166	0.189	0.88	30
	8-storeys	0.106	0.113	0.94	22
	12-storeys	0.097	0.098	0.99	22

$S_{am,Collapse}$ - Median collapse capacity; $S_{a,MCE}$ - Spectral acceleration demand corresponding to Maximum Considered Earthquake; and CMR - Collapse margin ratio.

It can be observed that all the regular symmetric building models have CMR very close to each other, with an average CMR of about 1.24. The CMR reduces significantly for mass-eccentric buildings with an average reduction of 20%, 22% and 25% for 5%, 10% and 15% mass-eccentric buildings, respectively. This interesting observation shows that the median collapse capacity of mass-eccentric buildings is affected significantly when compared with the corresponding regular symmetric building, however the difference in median collapse capacities of buildings with different eccentricities has been found out to be insignificant. This observation can be attributed to the fact that with increase in mass eccentricity from 0% to 5%, the ductility demand at the flexible side increases significantly. With a further increase in eccentricity from 5% to 10% and 15% the ductility demand at the flexible side almost remains same as that in case of 5% eccentricity. A similar observation has also been made by Halabian and Birzhandi (2013). Further, the reduction in median collapse capacity is the highest for the 4-storied mass-eccentric buildings as compared to that in cases of the 8- and 12-storied mass-eccentric buildings (Table 4).

FRAGILITY FUNCTIONS

In order to assess the effect of accidental eccentricity on collapse probability, the fragility analysis is performed by post-processing the IDA results following the methodology suggested by Haselton et al. (2011). In this methodology, the median collapse capacity $(S_{am,Collapse})$ and corresponding logarithmic standard deviation (also called as record-to-record variability parameter, β_{RTR}) are directly obtained from the IDA results. Based on the recommendations from previous studies (Haselton and Deierlein, 2007; Liel et al., 2009) the logarithmic standard deviation (β_m) for modelling uncertainty is taken as 0.50. Both the record-to-record variability and modelling variability are combined using the square-root-of-sum-of-squares (SRSS) method (Liel et al., 2009; Haselton et al., 2011) in order to obtain the total variability (β_T).Table 5 presents the fragility curve parameters obtained by post-processing of IDA results. The observed variabilities in regular as well as irregular buildings are in the range of 0.6-0.7, which is in agreement with the past studies (FEMA P58, 2012).

Table 5: Fragility Curve Parameters for the Considered Building Models Obtained from IDA

Building Model	Building Height	β_{RTR}	β_m	β_T
Regular symmetric building	4-storeys	0.35	0.50	0.61
	8-storeys	0.48	0.50	0.69
	12-storeys	0.47	0.50	0.69
5% Mass-eccentric buildings	4-storeys	0.33	0.50	0.60
	8-storeys	0.41	0.50	0.65
	12-storeys	0.45	0.50	0.67
10% Mass-eccentric buildings	4-storeys	0.31	0.50	0.59
	8-storeys	0.49	0.50	0.70
	12-storeys	0.41	0.50	0.65
15% Mass-eccentric buildings	4-storeys	0.27	0.50	0.57
	8-storeys	0.47	0.50	0.69
	12-storeys	0.45	0.50	0.67

Figure 6 presents the fragility curves derived for incipient collapse of the considered building models. The corresponding collapse probabilities for DBE and MCE in the considered seismic zone, are presented in Table 6. It can be observed that the Indian code-designed regular symmetric buildings have collapse probabilities of the order of 9% and 37% for DBE and MCE demands, respectively. Expectedly, the mass-eccentric building models have a higher probability of collapse compared to the corresponding regular symmetric building. On average, mass-eccentric buildings have 16% and 53% probabilities of collapse for DBE and MCE demands, respectively. The highest effect of mass-eccentricity on collapse probability is observed for the 4-storied building and the least effect for the 12-storied building. Further, the obtained collapse probabilities are much higher than the limit prescribed by FEMA P695 (2009) which considers an average of 10% probability of collapse (for MCE) as acceptable for a group of buildings.

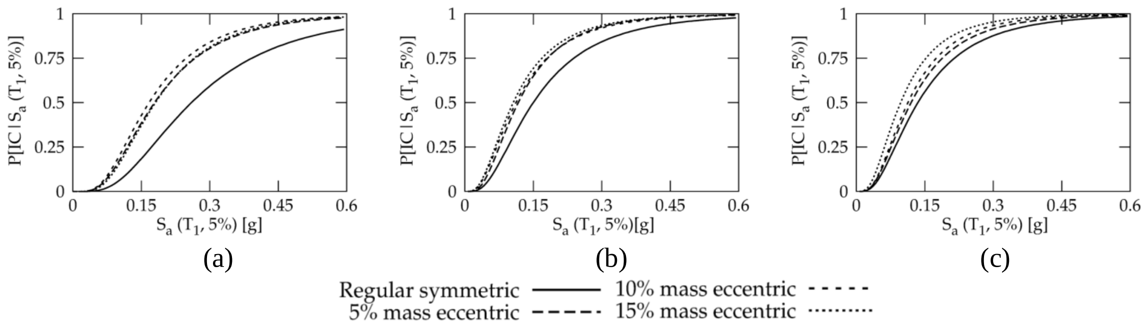

Fig. 6 Fragility curves for regular symmetric and mass-eccentric buildings: (a) 4-storey buildings, (b) 8-storey buildings, and (c) 12-storey buildings. (P [IC| S_a (T_1, 5%)] represents the probability of incipient collapse for a given spectral acceleration demand, S_a (T_1, 5%))

Table 6: Collapse Probabilities for the Considered Building Models

Building Model	Building Height	P [IC/DBE]	P [IC/MCE]
Regular symmetric building	4-storeys	06	36
	8-storeys	09	37
	12-storeys	11	38
5% Mass-eccentricbuildings	4-storeys	17	57
	8-storeys	15	52
	12-storeys	10	39
10% Mass-eccentricbuildings	4-storeys	19	62
	8-storeys	19	54
	12-storeys	11	43
15% Mass-eccentricbuildings	4-storeys	11	63
	8-storeys	21	55
	12-storeys	16	52

P[IC/DBE] and P[IC/MCE] represent the probability of incipient collapse for the Design Basis Earthquake and the Maximum Considered Earthquake, respectively.

CONCLUSIONS

The effects of varying accidental eccentricity on the seismic response of multistoried RC frame buildings, their collapse margin ratio, and collapse probabilities has been studied using incremental dynamic analysis. Twelve mid-rise RC frame buildings with 4-, 8- and 12-storeys have been studied. All the buildings were designed for the regular symmetric case while accidental eccentricity was introduced by changing the distribution of mass keeping the total mass at a floor level the same as corresponding to a regular symmetric building.

It has been observed that 5% accidental eccentricity results in maximum to average inter-storey drift ratios greater than 1.2, as well as maximum to minimum displacement ratios greater than 1.5, for all building models. The considered mass-eccentric building models have been found to be torsionally irregular as per IS 1893 (both 2002 and 2016 versions) and ASCE 7 whereas torsionally regular according to NZS 1170.5 and EN 1998. Only buildings with 15% eccentricity are considered to be torsionally irregular as per EN 1998. This observation highlights the significant variation existing among the different national seismic design codes in the definition of torsional irregularity. Further, the sole consideration of 5% accidental eccentricity, even for a perfectly symmetric building, leads to classify it as a torsionally irregular building. This illustrates that the definition of torsional irregularity requires further investigations.

The median collapse capacity obtained for regular symmetric and mass-eccentric building models shows that mass eccentricity leads to a reduction in median collapse capacity of the buildings. It is observed that 15% accidental eccentricity results in a reduction in median collapse capacity up to 33% for the 4-storied building while this reduction in median collapse capacity decreases with increasing building height. The obtained CMR for symmetric buildings are of the same order while they increase with increasing building height for mass-eccentric building models.

The fragility analysis of the considered building models suggests that even regular symmetric buildings have high probabilities of collapse of the order of 9% and 37% for DBE and MCE demands, respectively. These probabilities further increase to 16% for DBE and 53% for MCE due to accidental torsional effects. The presented results highlight the importance of accidental eccentricity in the seismic design of buildings and call for a risk-based design framework in order to reach an acceptable collapse performance.

It is important to note that the accidental eccentricity may arise due to other unforeseen factors that are not considered herein. Hence, the conclusions derived herein are strictly applicable to buildings where accidental eccentricity occurs due to mass eccentricity. Further, the investigated building models have rectangular plan shapes and therefore the observations and conclusions drawn in this study are limited to similar buildings only.

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