[redacted]

9-10 September 2019, Greenwich, London

COMPARATIVE SEISMIC FRAGILITY OF TORSIONALLY IRREGULAR RCC BUILDINGS DESIGNED USING INDIAN AND EUROPEAN CODES

Payal GWALANI[1], Yogendra SINGH[2] & Humberto VARUM[3]

Abstract: *Presence of irregularities in building tends to increase its seismic vulnerability. To improve their performance, many current seismic design codes specify larger demand and more stringent design requirements for torsionally irregular buildings, as compared to regular buildings. The current Indian seismic design code (IS 1893 Part 1), however, provides design guidelines that are essentially applicable to only regular buildings. With this view, the objective of the present study is to evaluate the behaviour and capacity at collapse of mid-rise RC frame-shear wall buildings, with and without torsional irregularity, designed according to current IS 1893 (Part 1) and BS EN 1998-1 codes. For this purpose, three-dimensional building models are subjected to bi-directional incremental dynamic analyses (BIDA), using a set of far-field ground motion records. To account for the cyclic deterioration of stiffness and strength, the non-linear behaviour of beams is modelled using an experimentally calibrated lumped plasticity model, while the columns and shear-walls are modelled using fibre-hinge models (ETABS-CSI, 2016) duly calibrated with the experimental results available in literature. Results of the BIDA are used to assess the collapse capacity, and for developing seismic fragility curves according to the FEMA P695 methodology. The results are compared and discussed, with particular emphasis on the adequacy and limitations of the design provisions and recommendations in the two codes, in context of torsionally irregular buildings.*

Introduction

Structural irregularities in the buildings are common, as the configuration has to meet the functional requirements of providing larger space, flexible/mixed usage, enhanced ventilation, better scenic views, and most importantly aesthetics. The asymmetric distribution of mass, stiffness and/or strength in the plan of the building results in torsional coupling, where the building experiences both lateral displacement and floor rotation. The seismic vulnerability of such torsionally irregular buildings has been repeatedly demonstrated in the past during several earthquakes (1971 San Fernando earthquake, 1989 Loma Prieta earthquake, 1994 Northridge earthquake, 1995 Kobe Earthquake, 2010 Maule Earthquake, 2011 Sikkim earthquake). A number of studies have been carried out in the past to study the seismic response of torsionally irregular buildings and to improve the design code provisions for them. Most of these studies were based on the use of single-storey simplified shear beam models (Chopra and Goel 1991, Zhu and Tso 1992, Chandler & Duan 1994, Tso and Wong 1995).

The recent advances in the computational tools have facilitated study of effect of torsional irregularities using more realistic 3D multi-storey non-linear models. These models are capable of representing more realistic distributions of stiffness and strength in plan and elevation. Several authors (Stathopoulos and Anagnostopoulos 2002, 2003; De Stefano et al. 2006) studied the response of realistic multi-storey models and pointed out the shortcomings of observations made from the simplified one-storey models. These studies clarified the contradictory observations regarding critical elements being on the flexible or stiff side of the torsionally irregular building. A limited number of studies have also been carried out to evaluate the collapse performance of torsionally irregular buildings. Manie et al. (2015) observed that collapse safety margin reduces as the amount of plan eccentricity and number of stories increases. Han et al. (2017) observed

[1] Research Scholar, Department of Earthquake Engineering, IIT Roorkee, Roorkee, India, payal.gwalani08@gmail.com

[2]Professor, Department of Earthquake Engineering, IIT Roorkee, Roorkee, India, yogendra.eq@gmail.com

[3]Full Professor, CONSTRUCT-LESE, Department of Civil Engineering, Faculty of Engineering of the University of Porto, Portugal, hvarum@fe.up.pt

that the collapse probability of torsionally irregular frames designed without additional requirements given in ASCE 7-10 was the highest, while irregular frames designed according to ASCE 7-10 requirements had lower probability of collapse in comparison with the regular frames designed for the same code. A detailed study based on the collapse safety evaluation of irregular buildings (including torsional irregularity) designed according to ASCE 7-16 seismic provisions has been presented in FEMA P-2012 (2018).

The focus of the present study is on evaluation of torsional design provisions given in IS 1893 (Part 1) and BS EN 1998-1, by comparing the collapse capacity of torsionally irregular buildings with the baseline model which is symmetric in plan and regular in elevation. For this purpose, two 8-storey frame-shear wall building models have been considered, one with torsional irregularity and other with torsional flexibility (but symmetric). Bi-directional Incremental Dynamic Analysis (BIDA) has been performed on the three-dimensional (3D) inelastic building models using far-field ground motion record suite. The results are expressed in terms of the dynamic capacity curves and fragility curves for the collapse damage state.

Comparison of Code Provisions

The code provisions for torsionally irregular buildings can be compared on the basis of definition of torsional irregularity, torsional flexibility, accidental eccentricity, type of analysis and additional design requirements. The Indian code IS 1893 (Part 1) defines a structure to be torsionally irregular, if the maximum storey drift at one end of the structure is more than 1.5 times the minimum storey drift at the other end of the same storey. When the drift ratio is greater than 2.0, the structure is considered to have extreme torsional irregularity. While BS EN 1998-1 defines torsional irregularity in terms of the ratio of stiffness eccentricity (e) and torsional radius (r). According to BS EN 1998-1 if the ratio of e/r is greater than 0.3, the structure is termed as torsionally irregular. A structure has torsional flexibility according to IS 1893 (Part 1) if the fundamental torsional mode period is greater than the two fundamental translational mode periods. BS EN 1998-1 defines torsionally flexible structure as the one in which the torsional radius (r) is smaller than the radius of gyration (l) of the floor mass in plan.

The accidental eccentricity which arises due to torsional component of ground motion and discrepancies in estimation of centres of mass and stiffness, is usually expressed as a fraction of the plan dimension (b) perpendicular to the direction of excitation. Both, IS 1893 (Part 1) and BS EN 1998-1 consider 5% accidental eccentricity i.e., i.e. 5% of the plan dimension perpendicular to the direction of the excitation. Both IS 1893 (Part 1) as well as BS EN 1998-1 recommend 3D dynamic analysis for torsionally irregular buildings, whereas the rest of the seismic design process remains same as that for regular structures. However, in case of torsionally flexible buildings, the BS EN 1998-1 reduces the behaviour factor to 2 for DCM and 3 for DCH class of buildings, whereas the IS 1893 (Part 1) does not have any change in the behaviour factor (termed as response reduction factor, R in IS 1893).

Numerical Study

In this study, 8-storey RC frame-shear wall residential building models with plan dimension 16 x 13 m, as shown in Figure 1, have been considered. Three types of building models have been considered, namely:

1. **Symmetric-regular:** The baseline building model (as shown in Figure 1(a)) with shear walls oriented along the perimeter.
2. **Torsionally flexible:** Torsional flexibility has been introduced in the baseline model by moving the shear walls towards the centre of plan, thereby reducing the torsional rigidity of the system (as shown in Figure 1(b)). The irregularity in such models is due to flexibility alone, with no inherent torsion.
3. **Torsionally irregular:** This model has eccentricity in one direction caused by asymmetrically shifting the position of shear walls (as shown in Figure 1(c)).

Modelling and Linear Analysis

The considered buildings have a constant storey height of 3.3 m and plinth height of 1.5 m. The 3D modelling and analysis of the buildings have been carried out in the proprietary software ETABS-CSI (2016). Beams and columns have been modelled with frame (line) elements. Shear walls have been modelled using the wide-column analogy, in which the equivalent column section has been assigned at the centre of the wall section and connected to the neighbouring elements using horizontal rigid links. Slab has been modelled as rigid diaphragm. In order to compare only the torsional irregularity provisions of the two seismic codes, identical gravity and seismic actions have been considered in all the buildings (in accordance with the relevant Indian codes in the present study). The characteristic strength of concrete (95% confidence cube crushing strength, f_{ck}) has been considered as 40MPa and the yield strength of steel reinforcement (f_y) as 500MPa has been used. Dead and live loads have been assigned in accordance with the provisions of IS 875 (Part 1) and IS 875 (Part 2), respectively. The RC buildings considered have a standard occupancy (Importance factor - 1) and are situated in the highest seismic zone (Zone V) of IS 1893 (Part 1) with an Effective Peak Ground Acceleration corresponding to the Maximum Considered Earthquake, having 2475 years return period, $EPGA_{MCE}$ of 0.54g (= 1.5 x Zone Factor). The supporting soil condition is hard/rock soil.

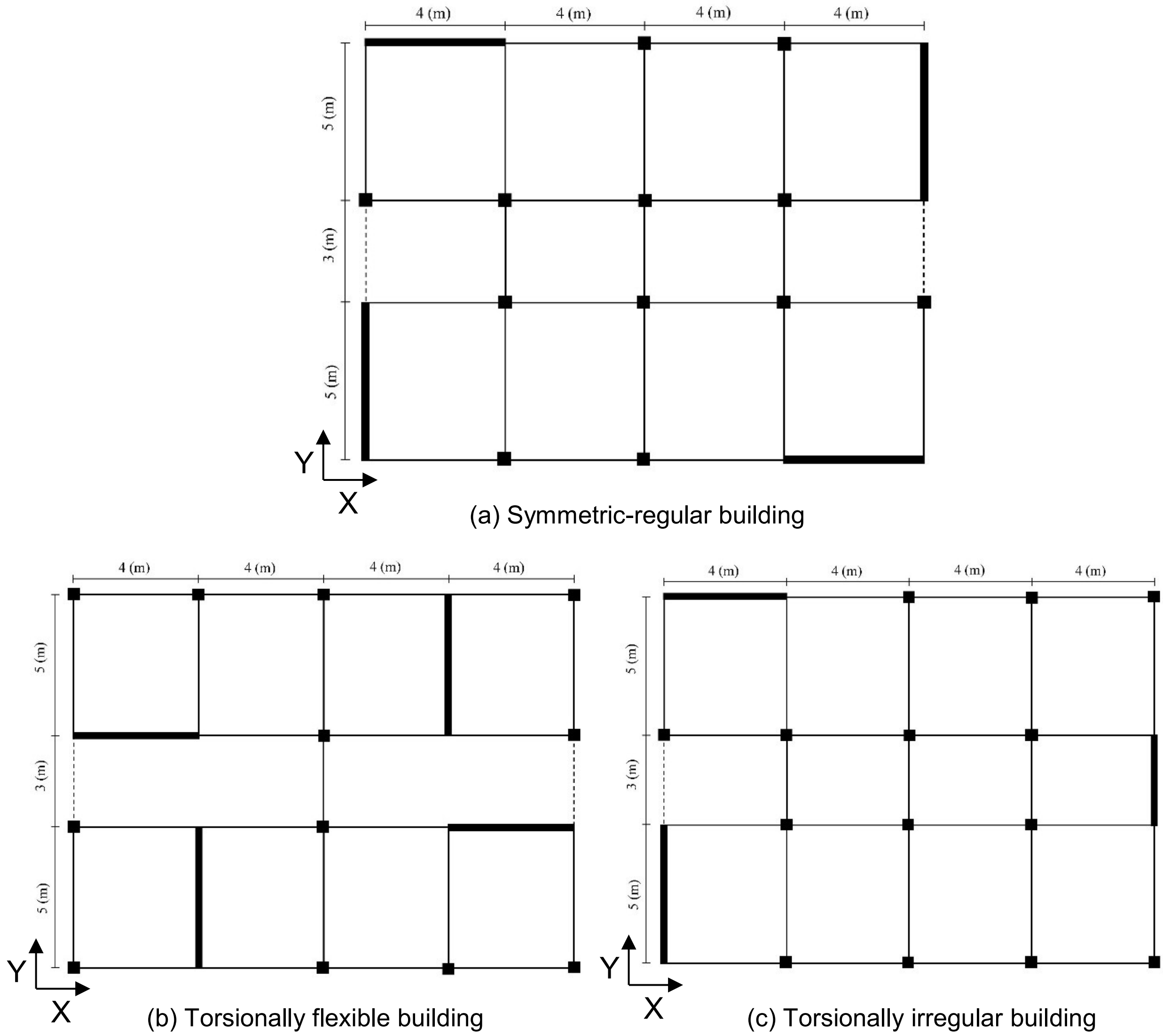


Figure 1. Plan of the considered building models.
(*All dimensions are in meters.*)

The building models have been designed as special moment resisting frame (SMRF), with ductile detailing (DCH in BS EN 1998-1) of structural members (beams and columns) and capacity design to avoid shear failure according to IS 13920. In the absence of the capacity design provisions of shear wall in the IS 13920, the provisions of BS EN 1998-1 have been followed to avoid the shear failure of RC shear walls. In case of the building models, where shear wall takes up most of the base shear (>75%, e.g. in case of the Symmetric-regular building), the moment frame has been designed to resist a minimum 25% of the base shear force. The beams and columns have been designed to have a strong column-weak beam ratio of 1.4 according to IS 13920 recommendations. The cracked section properties of structural members have been considered in accordance with IS 1893 (Part 1) guidelines. The second order (P-delta) effects have been considered in the analysis and design of the models. It is difficult to consider the accidental torsional effects in the 3D dynamic analysis and therefore in the present study these have been considered in the design using static analysis for an additional torsional moment corresponding to the displacement of centre of mass by a distance of ±0.05*b*.

A critical task while determining the torsional irregularity coefficient (*e*/*r*) according to BS EN 1998-1 is the calculation of stiffness eccentricity and torsional radius. BS EN 1998-1 specifically does not provide any method for calculating these values. In the present study, a single floor definition of the centre of rigidity has been considered, and the stiffness eccentricity, and lateral and torsional stiffnesses are determined at each storey level using the methodology described in Bisch et al. (2012). The dynamic properties obtained from the modal analysis of the building models have been shown in Table 1. The table shows the mass participation in each principal direction and the maximum value (of all the storeys) of torsional irregularity coefficients, viz. $\Delta_{max}/\Delta_{min}$, *e*/*r*, and *r*/*l*. It can be observed from the table that the irregularity classification (Symmetric-regular/Torsionally flexible/Torsionally irregular) of the considered building models using both the codes is matching.

Building Model	Mode Number	Period of vibrati-on	Modal mass participation ratio		Regularity check					
					X-direction			Y-direction		
		T (s)	α_{mx}	α_{my}	*e*/*r**	*r*/*l*#	$\Delta_{max}/\Delta_{min}$*	*e*/*r**	*r*/*l*#	$\Delta_{max}/\Delta_{min}$*
Symmetric-regular	Mode 1	1.71	0.67	0.00						
	Mode 2	1.64	0.00	0.65	0.00	0.00	1.00	0.00	0.00	1.00
	Mode 3	1.28	0.00	0.00						
Torsionally flexible (IS)	Mode 1	1.93	0.66	0.00						
	Mode 2	**1.92**	**0.00**	**0.00**	0.00	**0.72**	1.00	0.00	**0.58**	1.00
	Mode 3	1.65	0.00	0.65						
Torsionally flexible (EC)	Mode 1	1.88	0.69	0.00						
	Mode 2	**1.87**	**0.00**	**0.00**	0.00	**0.70**	1.00	0.00	**0.57**	1.00
	Mode 3	1.66	0.00	0.66						
Torsionally irregular	Mode 1	2.45	0.00	0.68						
	Mode 2	1.69	0.67	0.00	0.00	1.43	1.00	**0.45**	2.55	**1.79**
	Mode 3	1.54	0.00	0.00						

e - eccentricity; *r* - torsional radius; *l* - radius of gyration; α_{mx} and α_{my} denote the modal mass participation ratio in the *x* and *y* direction, respectively. The values of regularity coefficients in bold indicate torsional irregularity in the building model.
*represents the maximum value encountered amongst the storey levels in the building model.
#represents the minimum value encountered amongst the storey levels in the building model.

Table 1. Dynamic characteristics of the considered building models.

In case of the considered symmetric-regular and torsionally irregular building models, the behaviour factor (*q*) specified by BS EN 1998-1 is same as the *R* value recommended by IS 1893 (Part 1). Therefore, there is no difference between the design actions according to the two codes for both the buildings. However, the behaviour factors in case of torsionally flexible building differ for the two codes. While the BS EN 1998-1 provides a reduced value of behaviour factor (compared to regular building) for torsionally flexible building, there is no such provision in IS 1893 (Part 1). Hence, the flexible building models designed according to Indian and Eurocode have been differentiated as ‘torsionally flexible (IS)’ and ‘torsionally flexible (EC)’ for design according to IS 1893 (Part 1) and BS EN 1998-1, respectively. The bi-directional components of the ground motion have been combined using 100% - 30% rule. The sizes of structural members have been selected such that the reinforcement ratio lies in the range of 1 - 3% for columns, 0.8 - 1.2% for beams and 0.8 - 2% for the shear walls. The selected member sizes for the different models have been presented in Table 2.

Non-linear Modelling

The non-linearity has been incorporated in the beam elements using lumped-plasticity model (i.e., uniaxial M3 moment hinges). The deformation-controlled backbone curve for M3 hinges has been modelled according to ASCE 41-13. The backbone curve inherently accounts for the strength deterioration effects, while the stiffness deterioration effects have been incorporated using the energy-based hysteresis model available in ETABS-CSI (2016). This model has been calibrated for beams, designed according to Indian standards, in the past study by Surana et al. (2017). The parameters corresponding to the ductile beams have been used in the present study. The non-linearity in case of columns and shear walls has been modelled using the fibre-hinge (fibre P-M-M) model (ETABS-CSI, 2016). In this model, fibre-hinge is assigned at the centre of the plastic hinge length measured from the faces of the connected elements. At each fibre-hinge, the section is discretised into a number of fibres, each representing either confined concrete, unconfined concrete or a reinforcing steel bar. In the present study, the material constitutive law for reinforcing steel is based on bilinear elastic-plastic material model with kinematic strain-hardening. The stress-strain curve for confined and unconfined concrete is based on the constitutive law proposed by Mander et al. (1988). The cyclic deterioration effects have been incorporated in the model using energy-based hysteresis model. The parameters for this hysteresis model have been calibrated with the experimental results for columns (Rodrigues et al. 2013) and shear-walls (Dazio et al. 2009) available in the literature (results not presented here for brevity). Rayleigh damping of 5% has been assigned to the model at the fundamental period and the period in which the cumulative mass participation becomes 90%.

Building Model	Shear Wall	Columns	Beams
Symmetric-regular	5000 x 150/ 4000 x 150	400 x 400/ 350 x 350	300 x 400
Torsionally flexible (IS)	5000 x 150/ 4000 x 150	400 x 400	300 x 400
Torsionally flexible (EC)	5000 x 175/ 4000 x 175	400 x 400	300 x 400
Torsionally irregular	5000 x 150/ 4000 x 150/ 3000 x 150	400 x 400/ 350 x 350	300 x 400

Table 2. Member sizes for the considered building models.
(All dimensions are in mm.)

Incremental Dynamic Analysis

Incremental Dynamic Analysis (IDA) consists in non-linear dynamic analyses of the building under increasing intensity of the ground motion, such that the structure goes from elastic to inelastic level and finally reaches instability level (Vamvatsikos and Cornell 2002). For any ground motion, the structure is considered to have been collapsed, if a slight increase in intensity measure (IM) results in an abrupt increase in the engineering demand parameter (EDP) or the EDP (such as the inter-storey drift) reaches its limit value. The reliability of the estimated collapse capacity depends to a large extent on appropriate selection and scaling of the ground motions. An ideal IM to represent the severity of the ground motion should result in the structural response with low variability, without being affected by other seismological parameters. The conventional IMs used in the IDA are peak ground acceleration (*PGA*) and spectral acceleration at fundamental period ($S_a(T_1)$). However, recent studies (Kazanti and Vamvatsikos 2015, Eads et al. 2015, Kohrangi et al. 2016) have shown that $S_{a,avg}$ (that represents the geometric mean of spectral acceleration in a suitable period range) is more efficient, resulting in structural response with very low variability. The use of a range of period in $S_{a,avg}$ includes the effect of period elongation and higher modes (especially important in case of irregular buildings) during inelastic analysis. Also, its use reduces the bias on the ground motion selection method (Kazanti and Vamvatsikos 2015, Eads et al. 2015).

In the present study, bi-directional IDA has been performed using the 22 far-field ground motion record suite recommended by FEMA P695 (2009). The two horizontal components of the ground motions have been applied simultaneously swapping the components in the two orthogonal directions, resulting in 44 dynamic capacity curves. $S_{a,avg}$ ($0.2T$-$3T$, 5%) has been chosen as the IM, which represents the geometric mean of 5% damped spectral accelerations of the two components in the interval $0.2T$ - $3T$, where T is the average of the fundamental translational periods of vibration of the building model. The scaling of the two components by the same value (i.e., geometric mean) maintains the relative ratio amongst them. Inter-storey drift (θ_{max}) has been selected as the EDP due to its robustness in representing the structural damage (Vamvatsikos and Cornell 2002). The results from the BIDA have been post-processed to evaluate the seismic fragility of the building models using the methodology of FEMA P695 (2009). Under this methodology, the median collapse capacity ($S_{a,avg}$ (C)) and the record-to-record variability (β_{RTR}) are directly obtained from the results of IDA. The total variability is obtained as the sum of square roots of record-to-record variability (β_{RTR}) and modelling variability (β_M). The latter takes into account the effect of prevailing construction practices, construction material, design and detailing provisions, and the robustness of the analytical model in simulation of collapse. Currently, the reliable estimates of modelling variability in context of Indian buildings are not available. Therefore, in this study, the modelling variability values have been obtained from the previous studies (Haselton and Deierlein 2007; Liel et al. 2009).

Results and Discussion

The results from BIDA are presented in the form of median, 16th percentile and 84th percentile dynamic capacity curves. The IDA curves shown in Figure 2 are plotted in terms of $S_{a,avg}$ (0.2T-3T, 5%) and inter-storey drift (θ_{max}). It has been observed that collapse capacity of torsionally irregular building (designed as per both IS and EC standards) and torsionally flexible (IS) building is very low compared to the corresponding symmetric-regular building. The typical damage pattern at collapse for these building models have been presented in Figure 3. The collapse in case of regular building (Figure 3(a)) occurs due to the flexural failure of shear wall and most of the beams. In case of torsionally flexible building (Figure 3(b and c)), the failure of beam elements is mostly limited to the core region, while in case of regular buildings it is spread throughout. For torsionally irregular building, the collapse occurs due to the flexure failure of shear wall and large number of beams and columns on the flexible side (Figure 3(d)). The collapse capacity of torsionally flexible (EC) building has been observed to be 1.2 times the collapse capacity of regular building. The larger collapse capacity of flexible (EC) building is due to the more stringent design requirement (reduced behaviour factor) in BS EN 1998-1 for torsionally flexible buildings.

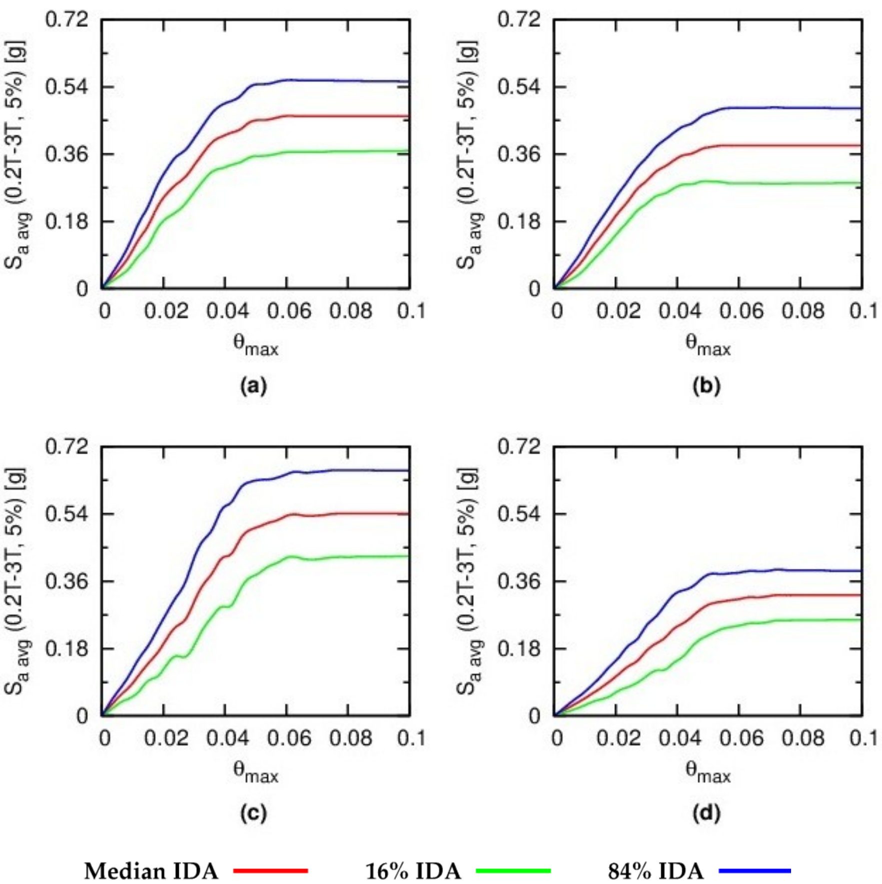
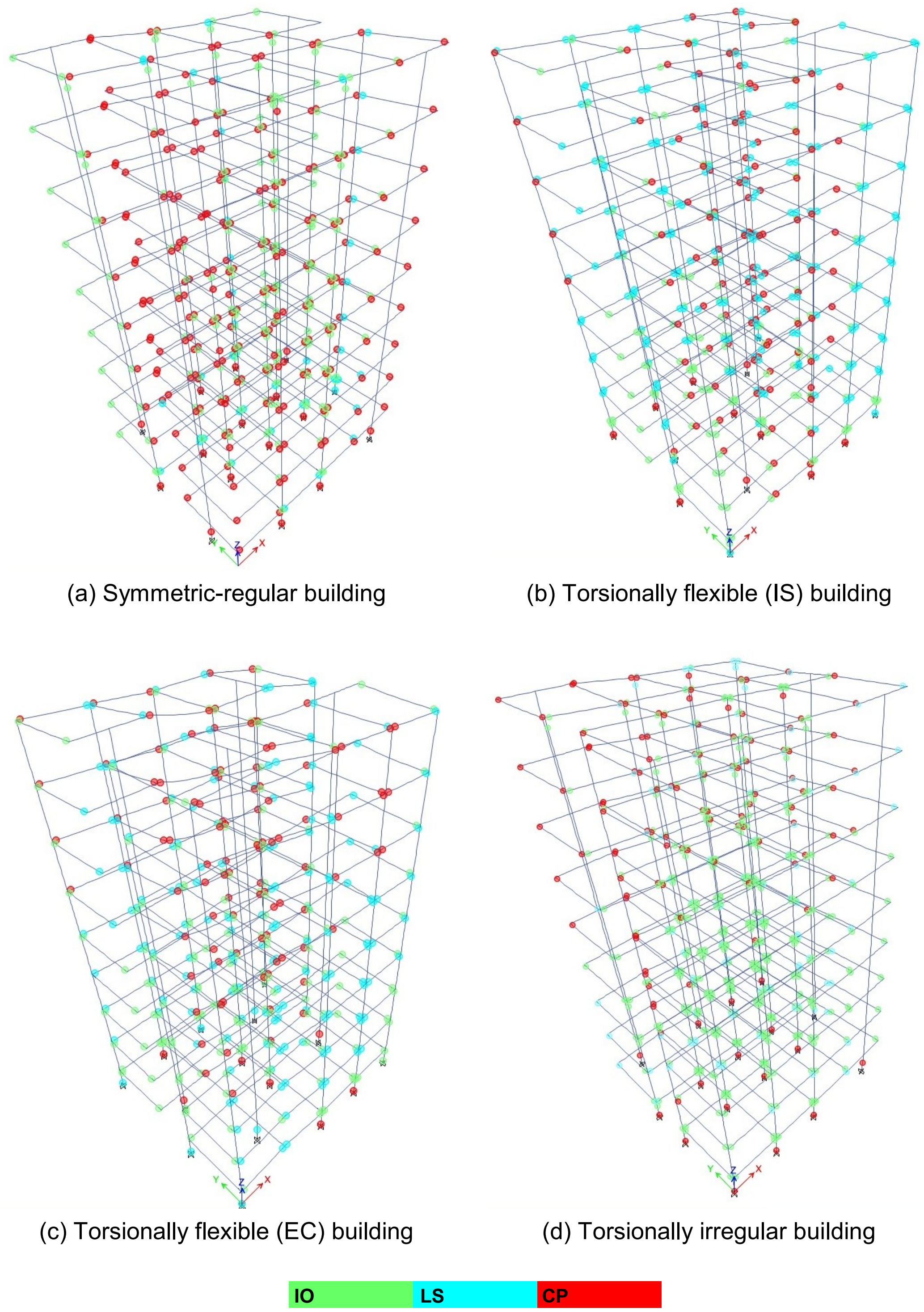


Figure 2. Dynamic capacity curves of the considered building models; (a) Symmetric-regular building; (b) Torsionally flexible (IS) building; (c) Torsionally flexible (EC) building and (d) Torsionally irregular building.



(a) Symmetric-regular building (b) Torsionally flexible (IS) building

(c) Torsionally flexible (EC) building (d) Torsionally irregular building

Figure 3. Failure pattern for the considered building models.

Table 3 presents the collapse margin ratio (CMR), defined as the ratio of median seismic collapse capacity ($S_{a,avg}$ (C)) to the seismic demand at MCE level ($S_{a,avg}$ (D_{MCE})), for the considered building models. The CMR serves as an indicator for comparing the relative collapse resistance of building models at MCE demand level. It has been seen that the collapse resistance of torsionally irregular building is smaller than the regular building. It can be noted that there is reduction in the collapse resistance of torsionally flexible (IS) building, even when it is torsionally regular. However, the collapse resistance increases considerably, when the additional design requirements of BS EN 1998-1, for torsionally flexible buildings are used.

Building Model	$S_{a,avg}$ (C)	$S_{a,avg}$ (D_{MCE})	CMR	% reduction in CMR
Symmetric-regular	0.47g	0.25g	1.89	-
Torsionally flexible (IS)	0.32g	0.23g	1.63	-13
Torsionally flexible (EC)	0.38g	0.24g	2.28	+20
Torsionally irregular	0.55g	0.22g	1.47	-20

Table 3. Collapse margin ratio of the considered building models.

The seismic collapse performance of the considered buildings has been evaluated in terms of fragility functions. Figure 4 presents the collapse fragility curves obtained from the median collapse capacity and variability parameters (Table 4) post-processed from the IDA results. The plots show the probability of collapse P[$C|S_{a,avg}$] of the building models in terms of the IM $S_{a,avg}$ ($0.2T$-$3T$, 5%) normalized with respect to the seismic demand at MCE i.e., $S_{a,avg}$ (D_{MCE}). The normalization of the IM with the seismic demand removes the bias due to the different translational periods of the building models. It has been observed that the collapse probability of regular building is slightly higher than 10% (but less than 20%), which is the limiting value for an archetype group, whereas those of torsionally flexible (IS) and torsionally irregular buildings are either close to or exceed 20%, which is the limiting value specified in FEMA P695 (2009) for individual buildings. In contrast, the probability of collapse of torsionally flexible (EC) building, designed using the additional design requirements of BS EN 1998-1 for flexible buildings lies well within the acceptable range (less than 10%). The results are summarized in Table 4.

Building Model	β_M	β_{RTR}	β_T	P[$C|S_{a,avg}$]
Symmetric-regular	0.50	0.16	0.50	11
Torsionally flexible (IS)	0.50	0.22	0.55	**19**
Torsionally flexible (EC)	0.50	0.17	0.52	06
Torsionally irregular	0.50	0.17	0.50	**23**

β_M - modelling variability; β_{RTR} - record-to-record variability; β_T - total variability, P[$C|S_{a,avg}$] - probability of collapse at MCE for a given $S_{a,avg}$. The values of P[$C|S_{a,avg}$] in bold are not acceptable according to FEMA P695 methodology.

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

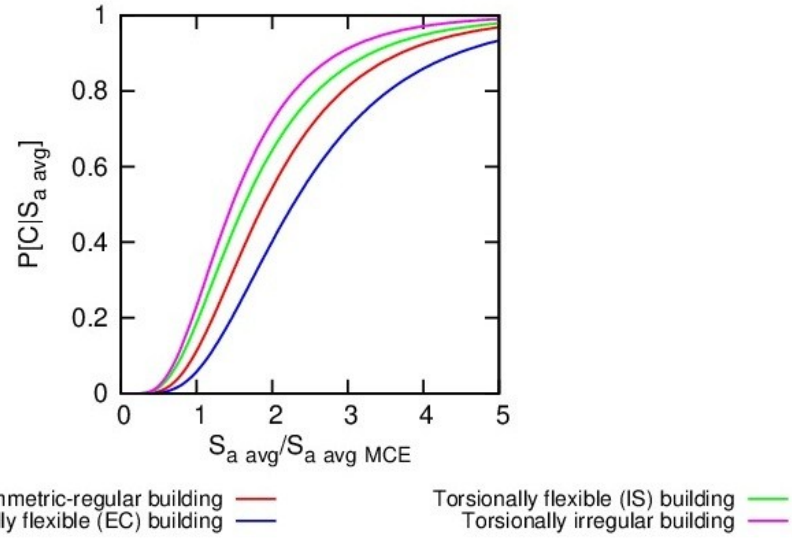


Figure 4. Fragility curves of the considered building models.

Conclusions

In order to quantify the effectiveness and to highlight the necessity of additional design requirements for torsionally irregular buildings, incremental dynamic analyses have been performed to determine the collapse resistance of archetypical building models designed using

Indian and European codes. Three 8-storey RC frame-shear wall building models with symmetric-regular, torsionally flexible and torsionally irregular configurations were designed according to both IS 1893 (Part 1) and BS EN 1998-1. It has been noted that the irregularity classification and design provisions for torsionally irregular buildings in both the considered codes match. Bi-directional incremental dynamic analyses have been conducted, and the collapse margin ratio and probability of collapse of each building model was evaluated. The fragility analysis of the investigated building models suggests that even symmetric building with torsional flexibility, designed as per Indian codes has a high probability of collapse of the order of 19% at MCE demand level. This probability further increases to 23% for MCE in case of torsionally irregular building, designed according to both IS 1893 (Part 1) and BS EN 1998-1. However, the torsionally flexible building designed using Eurocode outperforms the symmetric building, because BS EN 1998-1 requires use of reduced behaviour factors, in case of torsionally flexible building.

The presented results highlight the seismic vulnerability of torsionally irregular buildings and call for a need to improve the design provisions for torsionally irregular buildings in order to achieve an acceptable collapse performance. Based on the findings of this study, there is need for additional modifications to the behaviour factor of torsionally irregular buildings (with asymmetry in plan) in IS 1893 (Part 1) as well as in BS EN 1998-1. The results presented in this study are applicable to buildings with standard occupancy and typical loading conditions. To generalize the results obtained from this study, analyses of a number of building models with varying degree of torsional irregularities, structural configurations and height are required.

References

ASCE/SEI 41-13:2013, *Seismic evaluation and retrofit of existing buildings*, Reston, Virginia, United States

ASCE/SEI 7-10:2010, *Minimum design loads for buildings and other structures*, Reston, Virginia, United States

ASCE/SEI 7-16:2016, *Minimum design loads for buildings and other structures*, Reston, Virginia, United States

Bisch P, Carvalho E, Degee H, Fajfar P, Fardis M, Franchin P, Kreslin M, Pecker A, Pinto P, Plumier A and Somja H (2012), *Eurocode 8: Seismic design of buildings worked examples*, Luxembourg: Publications Office of the European Union

BS EN 1998-1:2004+A1:2013, *Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings*, AMD 31st May 2013

Chandler AM and Duan XN (1994), On the performance of inelastically responding asymmetric structures designed by the modal response spectrum method of NZS 4203:1992, *Bulletin of New Zealand National Society Earthquake Engineering,* 27(2): 96-106

Chopra AK and Goel RK (1991), Evaluation of torsional provisions in seismic codes, *Journal of the Structural Division*,117(12): 3762-3782

CSI (2016), *Integrated building analysis and design software (ETABS 2016)*, Computers and Structures Inc., Berkeley, California, United States

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

De Stefano M, Marino EM and Rossi PP (2006), Effect of overstrength on the seismic behaviour of multi-storey regularly asymmetric buildings, *Bulletin of Earthquake Engineering*, 4(1): 23-42

Eads L, Miranda E and Lignos DG (2015), Average spectral acceleration as an intensity measure for collapse risk assessment, *Earthquake Engineering and Structural Dynamics*, 44: 2057-2073

FEMA P2012:2018, *Assessing seismic performance of buildings with configuration irregularities*, Washington DC, United States

FEMA P695:2009, *Quantification of seismic performance factors*, Washington DC, United States

Han SW, Kim TO, Kim DH and Baek SJ (2017), Seismic collapse performance of special moment steel frames with torsional irregularities, *Engineering Structures*, 141: 482-494

Haselton CB and Deierlein GG (2007), *Assessing seismic collapse safety of modern reinforced concrete frame buildings*, PEER Report 2007/08, PEER Centre, University of California, Berkeley, CA

IS 13920:2016, *Ductile design and detailing of reinforced concrete structures subjected to seismic forces - Code of practice*, New Delhi, India

IS 1893-Part 1:2016, *Criteria for earthquake resistant design of structures: General provisions and buildings*, New Delhi, India

IS 875-Part 1:1987, *Code of practice for design loads (other than earthquake) for buildings and structures (Dead loads)*, New Delhi, India

IS 875-Part 2:1987, *Code of practice for design loads (other than earthquake) for buildings and structures (Live loads)*, New Delhi, India

Kazantzi AK and Vamvatsikos D (2015), Intensity measure selection for vulnerability studies of building classes, *Earthquake Engineering and Structural Dynamics*, 44(15): 2677-2694

Kohrangi M, Bazzurro P and Vamvatsikos D (2016), Vector and scalar IMs in structural response estimation, Part II: Building demand assessment, *Earthquake Spectra*, 32: 1525-1543

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

Mander JB, Priestley MJ and Park R (1988), Theoretical Stress-Strain Model for Confined Concrete, *Journal of Structural Engineering*, 114(8): 1804-1826

Manie S, Moghadam AS and Ghafory-Ashtiany M (2015), Collapse behavior evaluation of asymmetric buildings subjected to bi-directional ground motion, *The Structural Design of Tall and Special Buildings*, 24(8): 607-628

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

Stathopoulos KG and Anagnostopoulos SA (2002), Inelastic earthquake induced torsion in buildings: results and conclusions from realistic models, *In Proceedings of the 12th European Conference on Earthquake Engineering*, London

Stathopoulos KG and Anagnostopoulos SA (2003), Inelastic earthquake response of single-story asymmetric buildings: an assessment of simplified shear-beam models, *Earthquake Engineering and Structural Dynamics*, 32:1813-1831

Surana M, Singh Y and Lang DH (2017), Seismic characterization and vulnerability of building stock in hilly regions, *Natural Hazards Review*, 19(1): 04017024

Tso WK and Wong CM (1995), Eurocode 8 seismic torsional provisions evaluation, *Journal of European Earthquake Engineering*, 1: 23-33

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

Zhu TJ and Tso WK (1992), Design of torsionally unbalanced structural systems based on code provisions II: Strength distribution, *Earthquake Engineering and Structural Dynamics,* 21: 629-644