ISET Journal of Earthquake Technology, Paper No. 534, Vol. 54, No. 1, March 2017, pp. 17-29

COMPARATIVE ASSESSMENT OF SEISMIC FRAGILITY OF RC FRAME BUILDINGS DESIGNED FOR OLDER AND REVISED INDIAN STANDARDS

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ABSTRACT

Incremental dynamic analyses (IDA) are conducted on mid-rise (4- and 8-storey) special moment resisting RC frame buildings. The buildings are designed for the older and the revised Indian codes, without and with strong-column weak-beam (SCWB) design criterion, respectively. Three different configurations of RC frame buildings consisting of bare frame, uniformly infilled frame and open ground storey (OGS) are considered. In order to assess the seismic fragility, FEMA P695 methodology is used and median collapse capacity, collapse margin ratio (CMR) and record-to-record variability are estimated. The obtained CMR suggest that among the investigated buildings, the uniformly infilled and OGS buildings have better performance, when compared with their counterpart bare frames. The consideration of SCWB design leads to significant improvement in the seismic performance and collapse fragility of buildings, which reduces upto 16% and 50%, in case of 4- and 8-storey buildings, respectively.

KEYWORDS: Collapse Margin Ratio, Design Code, Incremental Dynamic Analysis, RC Frame Buildings, Seismic Fragility

INTRODUCTION

India is currently undergoing a rapid development, as a result of this, a large number of multi-storey buildings are coming up. In the past two decades, India has faced few devastating earthquakes which include 2001 Bhuj earthquake, 2011 Sikkim earthquake, and 2015 Nepal earthquake. Among these earthquakes, 2001 Bhuj earthquake has exposed the seismic vulnerability of the RC frame buildings, particularly, of uniformly infilled frame and open ground storey (OGS) buildings (Jain et al. 2002; Agarwal et al. 2002). It has been reported that a number of buildings has collapsed during 2001 Bhuj earthquake, particularly the uniformly infilled frames and OGS buildings. In India, the infilled frames are still extensively used for commercial as well as residential purpose even in high seismic zones, with OGS being mainly used for parking purpose.

Recently, the Indian seismic design and detailing codes both went under major revisions (IS 1893 Part 1 2016; IS 13920 2016). In the recent revision of the code (IS 13920 2016) additional requirement of strong-column weak-beam (SCWB) design has been included for seismic design of buildings. To prevent local collapse mechanism (failure of columns in a particular storey, also called 'weak storey mechanism') most of the seismic design codes, world-over, adopt strong-column weak-beam (SCWB) design. A review of the major international seismic design code provisions (i.e., ACI 318-14 2014; Eurocode 8 2004; NZS 3101 2006; IS 13920: 2016) reveals that significant variations exist in these code provisions in terms of the defined SCWB design requirements. ACI 318-14 (2014) suggests a SCWB ratio of 1.2 whereas Eurocode 8 (CEN, 2004) suggests a factor of 1.3. A more stringent approach is adopted by New Zealand's code (NZS 3101 2006) with a SCWB ratio of 1.3. In addition, NZS 3101 also applies a dynamic magnification factor for the upper floors, as these are significantly affected by higher mode effects. The dynamic magnification factor varies with building period (T) as well as along the height. In order to ensure, the SCWB design, a SCWB ratio (defined as the ratio of the sum of the nominal moment capacity of the columns to the sum of nominal moment capacity of the beams, both framing at the same joint in the direction under consideration) of 1.40 is recommended in IS 13920 (2016). This SCWB design is expected to have significant influence on the seismic performance of not only the RC bare frame buildings, but also in case of uniformly infilled and OGS buildings. It has already been shown that increasing column strength, without increasing the beam strength, is the most cost-effective measure to improve the seismic performance of the OGS buildings (Fardis and Panagiotakos 1997). In the same line, Eurocode 8 (2003) recommends increase in the strength of the columns alone of the open ground storey, to achieve the desirable performance.

Considering the observations from the previous study (Fardis and Panagiotakos 1997) and revised code provisions, in the preset study, seismic fragility of mid-rise (4- and 8-storey) RC frame buildings designed for the older (IS 1893 Part 1 2002; IS 13920 1993) and the revised Indian codes (IS 1893 Part 1 2016; IS 13920 2016) without and with SCWB design is assessed, for bare, uniformly infilled and OGS RC frame buildings. Incremental dynamic analyses (IDA) are conducted on the considered building models and the typical collapse mechanism, collapse margin ratio and collapse fragility are studied and compared.

MODELLING OF URM INFILLED RC FRAME BUILDINGS

In the present study, RC frame buildings with plan and elevation as shown in Figure 1 are considered. The building plan is chosen from a field survey to consider variety of characterisitcs of the building in National Capital Region (NCR) of India (DEQ 2009). The heights of these buildings are considered as 4- and 8-storeys, representing mid-rise building stock typical in NCR region. The storey height is taken as 3.3m, consistent with observations in field survey (DEQ 2009). The thickness of infill panels has been considered as 230 mm and 110 mm for exterior and interior panels, respectively. The compressive strength of infill panels (solid clay brick) is considered as 4.1 MPa considering the fair quality of masonry, consistent with typical compressive strength of masonry in Northern India. A total of 12 building models are investigated with two different design levels (without and with SCWB), two different heights (4- and 8-storeys) and three different configurations (viz. bare, uniformly infilled and OGS).

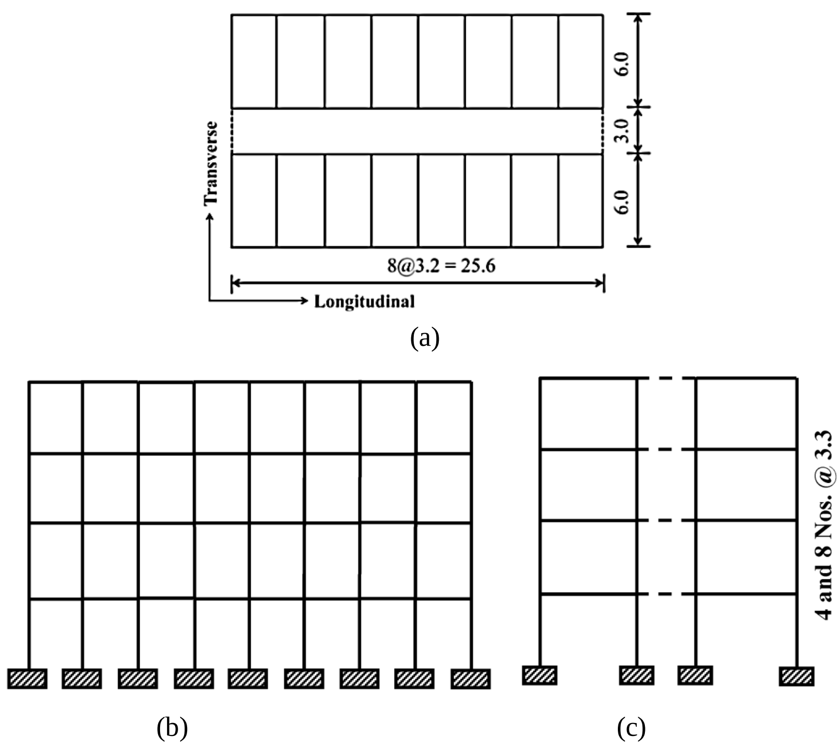


Fig. 1 (a) Generic plan, (b) front, and (c) side elevation of the considered buildings (The dotted lines represent the floor slabs boundaries, which are assumed to be rigid in its plane). All dimensions are in meter.

The buildings have been modelled in building analysis and design software ETABS (CSI 2016a, b). Beams and columns are modelled using 3-D frame elements while slabs have been considered as rigid diaphragms. The cracked section properties of beams and columns are derived following ASCE 41 (2013). Dead loads and live loads on the buildings have been assigned according to IS 875 Part 1 (1987a) and IS 875 Part 2 (1987b). In absence of the experimentally calibrated force-deformation behaviour of the masonry infills for Indian conditions, the modelling parameters from literature have been adopted. The eccentric strut model of ASCE 41 (2007) and Burton and Deierlein (2014) has been used to model infill panels. The initial (un-cracked) stiffness of the masonry infill panel has been taken as twice the stiffness obtained from the equivalent strut width model of ASCE 41, as recommended by Burton and Deierlein (2014) based on experimental investigations on infill panels. All the buildings are designed as Special Moment Resisting Frames (SMRF), following the Indian Standards IS 1893 (2016) and IS 13920 (2016) for seismic zone IV, situated on soil type I (i.e. hard soil/rock). All the considered building models are designed conforming to SCWB design criteria with typical SCWB ratio varying from 1.4 to 1.5. P-delta effect is also considered in analysis and design.

Table 1 presents the dynamic characteristics of the considered buildings obtained from the modal analysis as well as the periods obtained from empirical relationships provided in IS 1893 Part 1 (2002, 2016). It can be observed that a significant difference exists between two different estimates of the period of vibration. This difference between two estimates can be attributed to following facts: (i) the empirical formula for estimation of the fundamental period of vibration recommended in IS 1893 is to provide capping on design period (to ensure a minimum design force, irrespective of the stiffness assumption made by designer), and (ii) the fundamental period of vibration from modal analysis has been obtained considering the cracked section stiffness of the RC members which considers about 30%-70% of the gross-section moment of inertia as effective.

Table 1: Dynamic Characteristics of the Considered Building Models

Building configuration	Design level	No. of storeys	Fundamental period as obtained from modal analysis (s)		Design period as obtained from IS 1893 (s)		Modal mass participation factor (%)	
			Longitudinal direction	Transverse direction	Longitudinal direction	Transverse direction	Longitudinal direction	Transverse direction
Bare	Without SCWB	4	1.50	2.05	0.52	0.52	0.81	0.80
		8	2.49	3.99	0.87	0.87	0.79	0.78
Infilled		4	0.38	0.52	0.23	0.31	0.86	0.84
		8	0.72	1.03	0.47	0.61	0.81	0.79
OGS		4	0.82	0.90	0.23	0.31	0.99	0.98
		8	0.99	1.27	0.47	0.61	0.95	0.91
Bare	With SCWB	4	1.38	2.10	0.52	0.52	0.81	0.79
		8	2.32	3.85	0.87	0.87	0.79	0.78
Infilled		4	0.40	0.54	0.23	0.31	0.85	0.85
		8	0.71	1.00	0.47	0.61	0.82	0.79
OGS		4	0.77	0.91	0.23	0.31	0.98	0.97
		8	0.94	1.22	0.47	0.61	0.94	0.91

For nonlinear analysis, lumped plasticity model has been used to model the inelastic behaviour of the beams and columns. Flexural hinges (M3) and interacting hinges (P-M-M) are assigned at both the ends of beams and columns, respectively, and the corresponding hinge backbone curve parameters (Figure 2) are derived following ASCE 41 (2013) guidelines and presented in Table 2. These parameters in ASCE 41 are obtained from cyclic envelop, and thereby include strength deterioration effects. In order to consider stiffness deterioration under cyclic loading, pivot hysteresis model (Dowell et al. 1998) has been used. In this hysteretic model, the unloading and reverse loadings tend to be directed towards specific points called the pivot points, in the force-deformation plane. The corresponding hysteresis model parameters are adopted from CSI (2016a, b). The shear failure of columns has been modelled as per ASCE 41 (2013), and the model considers the effect of strut action of the infill on column shear.

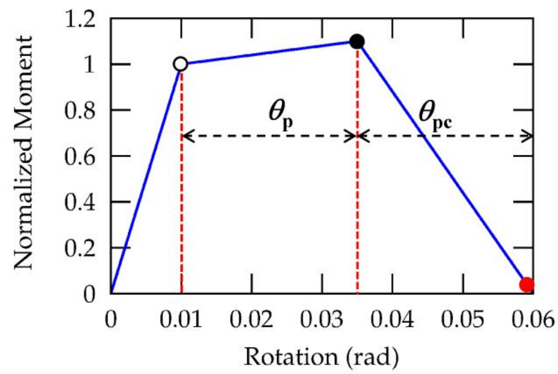


Fig. 2 Generalized force-deformation behaviour for RC frame elements

Table 2: Typical Modelling Parameters for Beams and Columns for Uniformly Infilled Frames

Design Level	No. of storeys	Member Description	Concrete compressive strength (MPa)	Rebar strength (MPa)	Member sizes (mm)	Axial force ratio	θ_p (Rad.)	θ_{pc} (Rad.)
Without SCWB	4	Beam	30	500	300 x 350	0.00	0.025	0.025
		Column	30	500	300 x 350	0.23	0.027	0.007
	8	Beam	30	500	300 x 400	0.00	0.025	0.025
		Column	30	500	400 x 400	0.52	0.027	0.007
With SCWB	4	Beam	30	500	250 x 350	0.00	0.025	0.025
		Column	30	500	350 x 350	0.20	0.026	0.007
	8	Beam	30	500	300 x 400	0.00	0.025	0.025
		Column	30	500	400 x 400	0.52	0.015	0.003

θ_p and θ_{pc} represent the pre- and post-capping plastic rotation capacities, respectively.

In order to model the nonlinear behaviour of infills, a number of models are available in literature (Klinger and Bertero 1978; Zarnic and Gostic 1997; Dolsek and Fajfar 2008; Burton and Deierlein 2014). Generally, infills have the two most prominent failure modes viz. shear failure and diagonal compression failure. The strength of infills is usually minimum in shear, and thereby the shear failure governs the inelastic modelling of infills. Based on the experiments conducted on the infill panels, Burton and Deierlein (2014) have proposed a three branch force-deformation curve for infills (Figure 3) which also includes post-peak behaviour of infills. The post-peak behaviour is particularly important in simulation of collapse (Burton and Deierlein 2014).

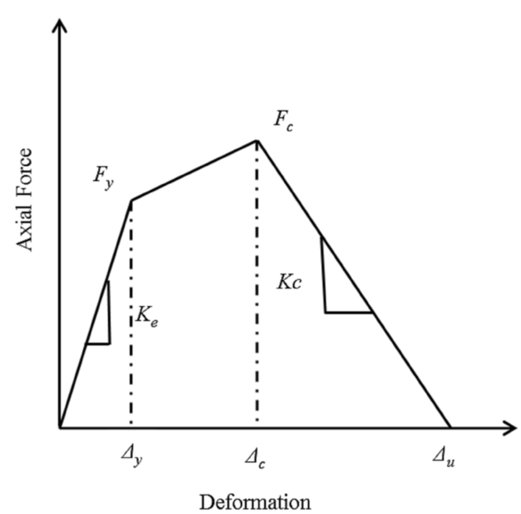


Fig. 3 Generalized force-deformation behavior of infill strut (Burton and Deierlein 2014)

The parameters, peak strength (F_c), yield strength (F_y), peak displacement $(\varDelta_c)$, yield displacement $(\varDelta_y)$ and ultimate displacement $(\varDelta_u)$ have been adopted from Burton and Deierlein (2014) and are presented in Table 3. The same force-deformation curve has been used in the present study. In addition, the ratio of the post-peak to the initial stiffness of 0.035 has been used in accordance with Burton and Deierlein (2014). It is important to consider here that the above mentioned parameters are obtained for monotonic loading, therefore do not account for the cyclic deterioration of strength and stiffness, which are particularly important to be considered near collapse (PEER/ATC 72-1 2010). Therefore, in order to consider the cyclic deterioration effects, the above parameters (both the strength and stiffness) are reduced by constant factors (lesser than unity) to obtain equivalent cyclic backbone curve directly from monotonic backbone curve, as recommended in PEER ATC 72-1 (2010) guidelines. In order to model the damping effects, 5% Rayleigh damping has been assigned for the periods corresponding to the fundamental mode and the mode resulting a total of 95% mass participation.

Table 3: Typical Modelling Parameters for Infill Panels, Duly Adjusted for Cyclic Deterioration Effects, Considered in this Study

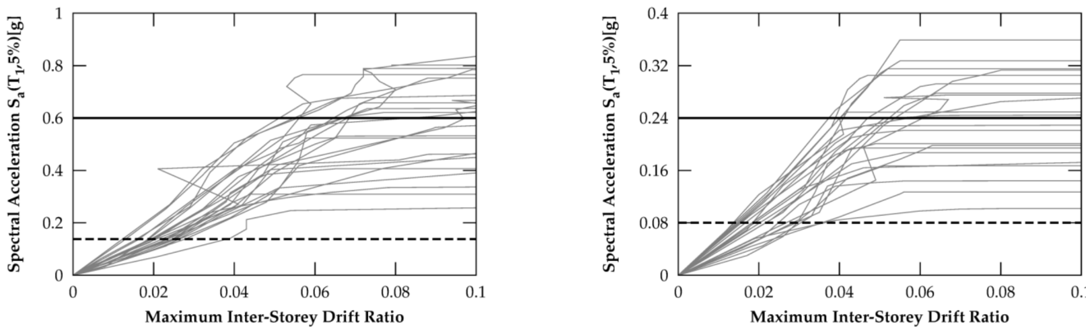
Design level	Direction	Thickness of infill panels (mm)	Aspect ratio of infill panels	F_y (kN)	F_c (kN)	$\varDelta_y$ (mm)	$\varDelta_c$ (mm)	$\varDelta_u$ (mm)
Without SCWB	Longitudinal	230	1.03	136.60	172.10	1.01	3.49	21.67
		110	1.03	68.30	86.00	0.94	3.25	37.19
	Transverse	230	0.55	210.90	265.67	1.52	5.27	32.75
		110	0.55	105.40	132.80	1.43	4.93	56.31
With SCWB	Longitudinal	230	1.03	135.5	170.70	0.94	3.24	20.16
		110	1.03	67.70	85.30	0.88	3.03	34.60
	Transverse	230	0.55	209.40	263.85	1.43	4.92	30.60
		110	0.55	104.70	131.90	1.33	4.60	52.49

INCREMENTAL DYNAMIC ANALYSIS

To investigate the nonlinear dynamic response of the considered buildings, Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell 2002) has been performed. In the present study, bi-directional IDA has been conducted using the 22 far-field ground-motion records, used in FEMA P695 project. The additional details of these ground-motion records can be found out in FEMA P695 (2009). The ground-motion records are applied in the two orthogonal building directions, simultaneously. To consider the bi-directional effects in ground-motion intensity measure, the geometric mean of spectral acceleration at the average period of the building is considered (FEMA P58 2012). Each ground-motion pair has been scaled in amplitude, until it causes structural collapse. In the present study, the buildings have been designed for capacity shear with SMRF detailing. These buildings are designed to ensure the ductile failure modes (flexural failure) resulting in side-sway collapse mechanism. Haldar and Singh (2012) have shown that shear failure of members and joints is not expected (FEMA P695 2009) in these buildings, even in case of infilled frames. However, in analytical models, both shear and flexural failure modes have been simulated; though, no shear failure has been observed in the numerical study. The structural collapse is defined as when a slight increase in amplitude of the ground-motion causes a very large increase in the inter-storey drift (Vamvatsikos and Cornell 2002; Haselton et al. 2011). From the IDA, the median collapse capacity and collapse margin ratio (CMR), (defined as the ratio of the median collapse capacity to the spectral acceleration demand corresponding to fundamental period at MCE level) have been estimated for each of the considered building model.

DYNAMIC CAPACITY CURVES

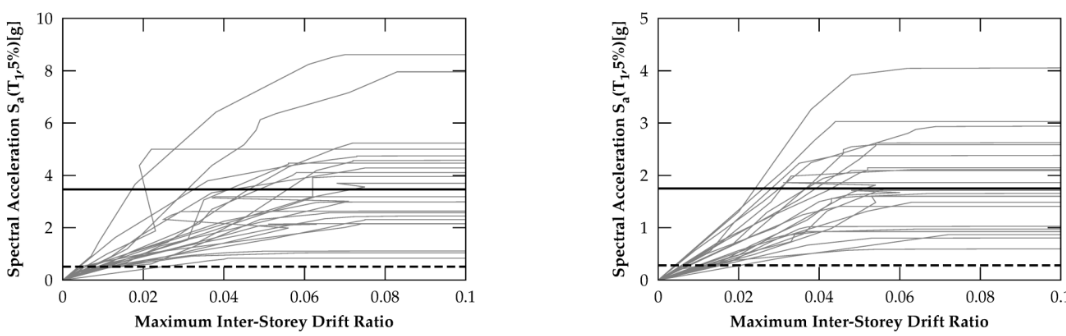
Figures 4 to 6 show the IDA curves, for the 4- and 8-storey bare frames, uniformly infilled frames and OGS buildings with SCWB design. Similar results have also been obtained for buildings without SCWB design, but are not presented for brevity. These IDA curves are plotted in terms of the 5%-damped spectral acceleration corresponding to the average period of vibration (average of the first mode periods in the two orthogonal building directions) versus the maximum inter-storey drift ratio. The firm horizontal line in these curves shows the median collapse capacity of the building, which is the spectral intensity, at which half of the ground-motions caused the structural collapse. The horizontal dotted line shows the 5%-damped spectral acceleration corresponding to the average period of vibration at MCE level, for the considered building. The ratio of spectral acceleration corresponding to these two lines represents the CMR.



(a) (b)

Median Collapse Capacity ———— Spectral Acceleration at MCE - - - - - - - - IDA Curve ————

Fig. 4 Dynamic capacity curves (for a suite of 22 far-field ground-motion records) of RC bare frame buildings designed as SMRF with consideration of SCWB ratio, as per relevant Indian Standards, for: (a) 4-storey, and (b) 8-storey



(a) (b)

Median Collapse Capacity ———— Spectral Acceleration at MCE - - - - - - - - IDA Curve ————

Fig. 5 Dynamic capacity curves (for a suite of 22 far-field ground-motion records) of uniformly infilled RC frame buildings designed as SMRF with consideration of SCWB ratio, as per relevant Indian Standards, for: (a) 4-storey, and (b) 8-storey

It can be observed that for a given building height, the uniformly infilled frame has the highest median collapse capacity, followed by the OGS, and the bare frame buildings. The reason for this observation can be attributed to the fact that the infilled frames were designed for the higher base shear coefficient (due to reduced design period, Table 1) as compared to bare frame buildings. Table 4 presents the CMR obtained for the building models investigated in the present study.

It can be observed that, in both the cases (without and with SCWB design), for a given building height, the uniformly infilled frame has the highest CMR whereas the bare frame has the lowest CMR, with the CMR of the OGS buildings, falling in between the two cases (except the case of 4-storey building without SCWB). In case of 4-storey building without SCWB, OGS building has the lowest CMR. Further, due to SCWB design, the CMR increases by 60% (on average) in case of 4-storey buildings, whereas in case of 8-storey buildings this increase is observed to be only 7% (on average). This observation can be attributed to the fact that the infills have similar strength in both the cases (without and with SCWB design), whereas the frames have different strengths. The relative increase in strength of frames after designing for a SCWB factor of 1.40 is relatively higher in case of 4-storey buildings as compared to 8-storey buildings.

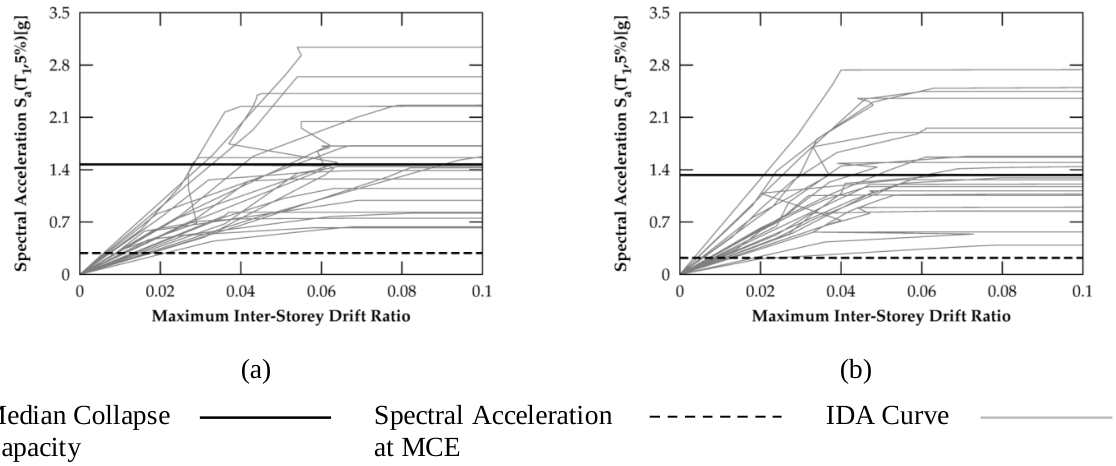


Fig. 6 Dynamic capacity curves (for a suite of 22 far-field ground-motion records) of open ground storey buildings designed as SMRF with consideration of SCWB ratio, as per relevant Indian Standards, for: (a) 4-storey, and (b) 8-storey

Table 4: Collapse Margin Ratio (CMR) for the Considered Building Models

Building configuration	Design level	No. of storeys	CMR
Bare frame	Without SCWB	4	3.10
Uniformly infilled			4.33
Open ground storey			2.95
Bare frame		8	2.70
Uniformly infilled			6.68
Open ground storey			5.24
Bare frame	With SCWB	4	4.59
Uniformly infilled			6.75
Open ground storey			5.16
Bare frame		8	3.09
Uniformly infilled			6.22
Open ground storey			5.97

The presented observations are significantly important which show that, in general, the uniformly infilled frames as well as OGS buildings have better performance as compared to bare frame, when designed for SCWB provision. This is mainly due to two reasons: (i) as mentioned earlier, the uniformly infilled frames and OGS buildings are designed for higher base shear coefficients due to shorter design periods, resulting in higher strength capacity; (ii) due to enhance SCWB ratio and SMRF design and detailing, the undesirable failure mechanisms (weak storey mechanism and shear failure of columns) in case of uniformly infilled as well as OGS buildings are avoided, resulting in adequate ductility.

Figures 7 to 9 present the dominant failure modes of 4-storey bare, uniformly infilled and OGS buildings. The failure modes shown are the collapse modes experienced most frequently, in a set of 22 far-field ground-motion records. The collapse mechanism of the bare frame usually involves yielding in beams and columns (Figure 7), and final collapse occurs due to failure of all the columns in either one or two storeys. This trend has been observed in 75% of the ground-motion records. In case of the uniformly infilled frame, first failure of infills takes place either in the first or second storey, followed by failure of beams and columns (Figure 8). This trend has been observed in 70% of the ground-motion records. In case of the OGS building also, the second storey infills fail before the failure of ground storey columns (Figure 9). This trend has been observed in 95% of the ground-motion records. Similar failure mechanisms have also been observed in case of 8-storey buildings, though the percentage of ground-motion records, leading to similar collapse mechanisms, differs marginally.

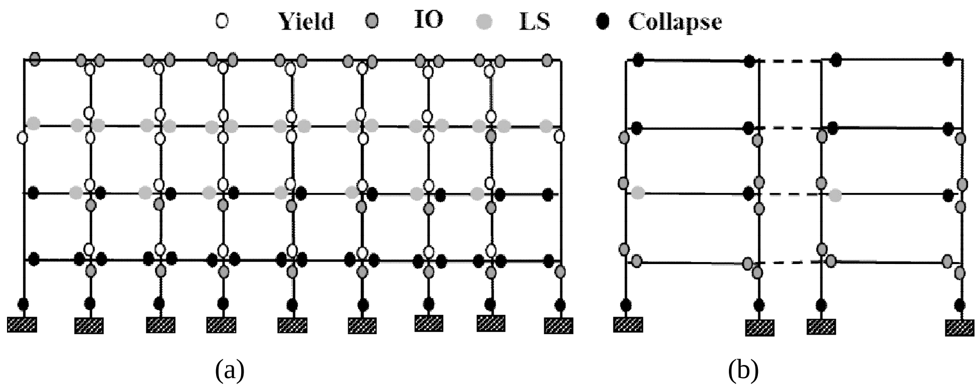


Fig. 7 Collapse mechanism of 4-storey RC bare frame building designed as SMRF with consideration of SCWB ratio, as per relevant Indian Standards: (a) typical longitudinal frame, and (b) typical transverse frame

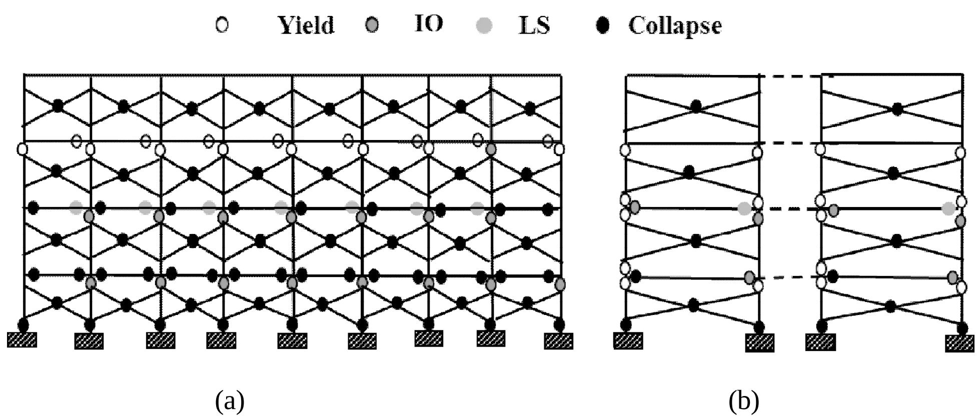


Fig. 8 Collapse mechanism of 4-storey uniformly infilled RC frame building designed as SMRF with consideration of SCWB ratio, as per relevant Indian Standards: (a) typical longitudinal frame, and (b) typical transverse frame

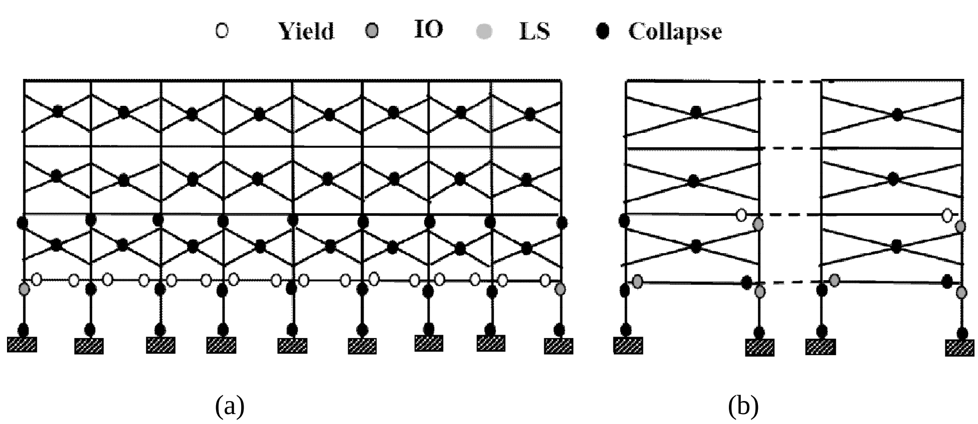


Fig. 9 Collapse mechanism of 4-storey open ground storey building designed as SMRF with consideration of SCWB ratio, as per relevant Indian Standards: (a) typical longitudinal frame, and (b) typical transverse frame

FRAGILITY ANALYSIS

The results obtained from IDA are post-processed to develop the collapse fragility curves. In the present study, the IDA has been conducted using a suite of 22 far-field ground-motion records, therefore, the record-to-record variability (β_{RTR}) has been computed directly, assuming that the collapse capacity follows a log-normal distribution (Haselton and Deierlein 2007). In addition, the modelling variability (β_m) of 0.50 is assumed (Liel et al. 2009) and combined with the obtained record-to-record variability (β_{RTR}) using Square Root of Sum of Squares (SRSS) technique as suggested in previous studies (Haselton et al. 2011).

Table 5 presents the obtained fragility curve parameters for the building models considered in the present study. It is observed from the table that the uniformly infilled and OGS buildings have higher record-to-record variability, when compared to their counterpart bare frames. This observation can be attributed to the fact that uniformly infilled and OGS buildings have relatively shorter period when compared with their counterpart bare frames. At short-period, the ground-motion records show higher variability; thereby, also increasing the variability in structural response of short-period buildings. This observation is found to be consistent with a previous study on non-ductile buildings (Sattar and Liel 2010).

Table 5: Fragility Curve Parameters for the Considered Building Models

Building configuration	Design level	No. of storeys	IDA results			
			Median collapse capacity (g)	Record-to-record variability (β_{RTR})	Modelling variability (β_m)	Total Variability (β_T)
Bare frame	Without SCWB	4	0.42	0.25	0.50	0.56
Uniformly infilled			2.30	0.62	0.50	0.79
Open ground storey			0.82	0.39	0.50	0.64
Bare frame		8	0.20	0.38	0.50	0.63
Uniformly infilled			1.83	0.51	0.50	0.71
Open ground storey			1.11	0.50	0.50	0.71
Bare frame	With SCWB	4	0.63	0.29	0.50	0.58
Uniformly infilled			3.46	0.59	0.50	0.77
Open ground storey			1.47	0.44	0.50	0.67
Bare frame		8	0.24	0.32	0.50	0.59
Uniformly infilled			1.75	0.48	0.50	0.69
Open ground storey			1.33	0.46	0.50	0.67

Figure 10 presents the collapse fragility curves for the considered 4- and 8-storey buildings, respectively. As the median collapse capacity, as well as the spectral acceleration demand, are functions of the building period, therefore, to compare the collapse fragility of the different buildings (i.e., bare, uniformly infilled and OGS), the fragility curves are presented in terms of normalized intensity (the spectral acceleration, normalized by the MCE level of seismic demand for seismic zone IV, corresponding to the average period of the building). Table 6 compares the collapse probabilities of the considered building models for MCE hazard. A comparison of the collapse probabilities of the buildings designed using the older codes (without SCWB design) and the revised codes (with SCWB design) shows that performance of the buildings improves significantly after SCWB design. The collapse probability of the buildings reduces up to 16% and 50%, after SCWB design in case of 4-storey and 8-storey buildings, respectively. It is interesting to note that the effect of SCWB design is more pronounced in case of infilled frame buildings than in case of bare frame buildings. This is due to avoiding of the weak-storey failure mechanism in case of buildings with SCWB design, as the strength of columns relative to the infills increases. Further, the effect is more pronounced in case of the 4-storey building than the 8-storey building. This observation can also be explained considering the relative strength of columns and infills. In the 4-storey as well as the 8-storey building, the infills remain the same, whereas the column sizes increase for the taller building. This reduces the chances of occurrence of the soft storey phenomenon in case of the 8-storey building, even without SCWB design, as evident from the collapse probabilities shown in Table 6.

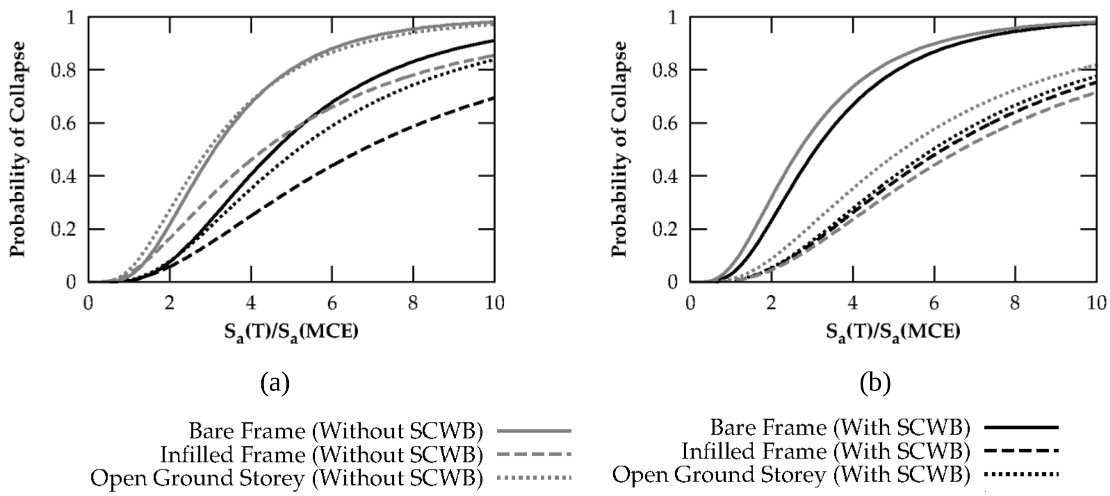


Fig. 10 Fragility curves for RC frame buildings considered in the present study (a) 4-storey, and (b) 8-storey

Table 6: Collapse Probability of the Considered Building Models, for MCE Demand

Building configuration	Design level	No. of storeys	Probability of collapse (%) at MCE
Bare frame	Without SCWB	4	2.50
Uniformly infilled			3.20
Open ground storey			4.70
Bare frame		8	5.70
Uniformly infilled			0.40
Open ground storey			1.00
Bare frame	With SCWB	4	0.50
Uniformly infilled			0.70
Open ground storey			0.80
Bare frame		8	3.20
Uniformly infilled			0.40
Open ground storey			0.50

In case of 4-storey buildings, the OGS buildings are the most vulnerable, whereas the bare frames are the least vulnerable, at MCE demand for both the design levels. (This is not very clear from Figure 10, as the considered demand lies in the lower tail of the fragility curves). This trend gets changed in case of 8-storey buildings, in which the bare frames are found out to be the most vulnerable and the uniformly infilled frames are found out to be the least vulnerable, at MCE. Interestingly, this observation is in partial contradiction with the CMR (as reported earlier), which shows uniformly infilled frames being the least vulnerable and the bare frames being the most vulnerable in both 4- and 8-storey buildings. The reason for this observation can be attributed to the fact that CMR only provides partial picture of the collapse performance as it does not account for the different variabilities in the seismic response. Further, the collapse fragility of the infilled frame reduces with increase in the building height, due to the fact that with the increase in building height, the strength of the frame members increases relative to the strength of infills.

The relatively superior performance of the infilled frames has also been observed in the past studies (Madan and Hashmi 2008). However, the superior performance of infilled buildings observed in the present study is in contrast to most of the previous studies (e.g., Haldar and Singh 2009). Interestingly, the SCWB design criterion does not only improve the seismic performance of the bare frame buildings, but it also improves the seismic performance of the uniformly infilled and OGS buildings, significantly. Irrespective of the building models considered in the present study, the collapse probability is found out to be well within the prescribed limit of 20% probability of collapse, conditioned on the occurrence of MCE, for individual buildings, as recommended in FEMA P695.

CONCLUSIONS

Seismic performance and fragility of 4- and 8-storey RC SMRF buildings without and with SCWB design following the older and revised Indian standards, respectively, have been assessed, using incremental dynamic analysis. The buildings have been considered with and without uniformly infilled frames and also with OGS. It has been observed that the consideration of SCWB design results in 60% increase in CMR for the considered 4-storey buildings, whereas only 7% increase in CMR has been observed for 8-storey buildings. The SCWB design results in a reduction in collapse probability up to 16% and 50%, in case of 4- and 8-storey buildings, respectively. The variable influence of the SCWB design of bare and infilled frames and buildings with different heights can be explained considering the strength of columns relative to infills. In case of infilled frames, the SCWB design has more dramatic effect due to avoiding of undesirable weak storey failure mechanisms, whereas, the effect reduces with increasing height of the building due to relatively lower increment in the strength of columns.

It has been observed that for a given building height, the uniformly infilled frame has the highest median collapse capacity, followed by the OGS, and the bare frame buildings. It is due to the higher design base shear coefficient in case of uniformly infilled frame as compared to the bare frame building. The bare frames, despite strong-column weak-beam design, have shown the combined beam-column failure mechanism, whereas in case of the uniformly infilled frames, the infills, either in the first or second storey, fail first, followed by failure of beams and columns. In case of OGS buildings, the infills in the second storey fail before the failure of ground storey columns.

The results of performance evaluation through CMR and fragility analysis are in contradiction for the 4-storey building, but in agreement in case of the 8-storey building. It has been observed that the uniformly infilled frames have the highest CMR and bare frames have the lowest CMR. The fragility analysis results show that in case of 4-storey buildings, the OGS building is the most vulnerable, whereas in case of 8-storey building, the bare frame is the most vulnerable. The reason for this difference in the two different performance measures lies in the fact that CMR does not account for the variability in the seismic response, therefore, it provides only partial picture about the collapse performance. The performance of all the building models investigated in this study has been observed to be satisfactory when compared with FEMA P695 performance criterion on the acceptable collapse probability.

The present study has been conducted on mid-rise buildings and a far-field record suite. Therefore, the observations and conclusions drawn in this study are strictly applicable to the considered building heights and record suite.

ACKNOWLEDGEMENTS

The first and second author’s fellowships granted by the Ministry of Human Resource Development, Government of India allowed the conduct of the present study. The support received from Ministry of Human Resource Development, Government of India, is gratefully acknowledged.

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