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Effect of URM infills on seismic vulnerability of Indian code designed RC frame buildings

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Abstract: Unreinforced Masonry (URM) is the most common partitioning material in framed buildings in India and many other countries. Although it is well-known that under lateral loading the behavior and modes of failure of the frame buildings change significantly due to infill-frame interaction, the general design practice is to treat infills as nonstructural elements and their stiffness, strength and interaction with the frame is often ignored, primarily because of difficulties in simulation and lack of modeling guidelines in design codes. The Indian Standard, like many other national codes, does not provide explicit insight into the anticipated performance and associated vulnerability of infilled frames. This paper presents an analytical study on the seismic performance and fragility analysis of Indian code-designed RC frame buildings with and without URM infills. Infills are modeled as diagonal struts as per ASCE 41 guidelines and various modes of failure are considered. HAZUS methodology along with nonlinear static analysis is used to compare the seismic vulnerability of bare and infilled frames. The comparative study suggests that URM infills result in a significant increase in the seismic vulnerability of RC frames and their effect needs to be properly incorporated in design codes.

Keywords: URM infill; RC frame building; Indian seismic design code; seismic performance; seismic vulnerability

1 Introduction

Past earthquakes, particularly the 2001 Bhuj earthquake, which was the first large earthquake in an urban area of India, have exposed the seismic vulnerability of RC frame buildings with unreinforced masonry infills, which is the most common structural system for multistory buildings in India and many other countries. Although URM infills are known to interact with and modify the seismic behavior of frame buildings (Paulay and Priestley, 1992; Singh *et al*., 1998; Sahota and Riddington, 2001), the present code procedures do not adequately account for this effect. Inadequate guidelines for the design of such buildings have resulted in a huge stock of seismically deficient buildings. This stock of vulnerable buildings needs to be evaluated for effective planning of mitigation measures.

In the present study, an effort has been made to study the effect of URM infills on the seismic performance and fragility of code-designed RC frame buildings. For this purpose, two sets of four-story generic RC frame buildings have been studied using ASCE 41 (2007) and HAZUS-MH (2003, 2006a,b) methodologies. Seismic performance and fragility of the buildings have been compared with and without URM infills. Two design levels of buildings are considered. The first set of buildings is designed for gravity load only, without considering any seismic forces, which is representative of many existing buildings that do not comply with modern seismic codes. The second set of buildings is designed as per Indian seismic codes (IS 456, 2000; IS 875, Part 1 and Part 2, 1987a,b; IS 1893, 2002), including ductile detailing of IS 13920 (1993).

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2 Role of infills

Unreinforced masonry infills are commonly used in India for partitions in multistory RC frame buildings. In design practice, these infills are treated as nonstructural elements and their interaction with the frame is often ignored. Ignorance of the interaction between the infill and the frame generally does not affect the gravity load resisting system in which all the gravity loads are resisted by frame elements. However, the behavior of the structure under lateral loads is significantly affected by the presence of URM infills. Infills act as diagonal struts (Fig. 1) and failure modes of the infilled frame are governed by the interaction between the frame and the infills.

3 Indian seismic code criteria for URM infilled RC frames

Inclusion of infills in frames provides considerable

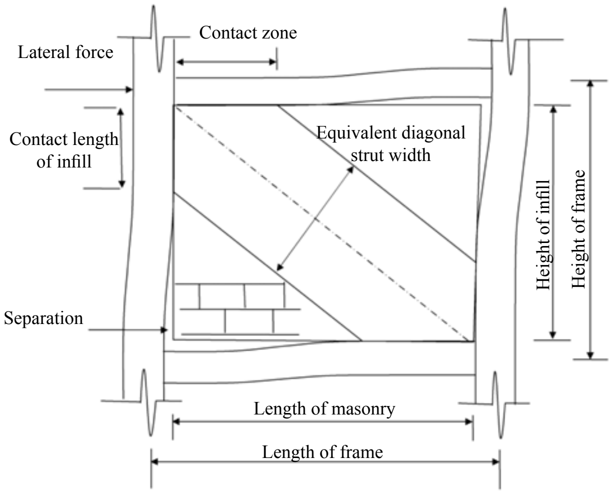

Fig. 1 Deformation of infilled frame under lateral load

additional stiffness resulting in a reduction of the natural period of vibration of the frame. The reduced natural period results in attraction of more force during earthquake. However, in practice, designers usually have a tendency to make flexible models of buildings, as it results in lower design base shear due to a lengthened period of vibration. To safeguard against this error, the code (IS 1893, 2002) has recommended a cap on the natural period for base shear calculation, to accommodate the increased stiffness due to infills.

The empirical expression for the design natural period for URM infilled RC frame buildings has been provided as

$$T_a = \frac{0.09h}{\sqrt{d}} \tag{1}$$

where T_a (s) is the design natural period of a building with a height equal to h (m) and base dimension d (m) along the direction of the vibration. The capping is implemented by scaling all the design member forces by a factor equal to $\frac{\overline{V}_B}{V_B}$, where $\overline{V}_B$ is the base shear calculated using the empirical design period and V_B is the base shear obtained analytically.

Further, it is common practice in India that the ground story is kept open in multi-story buildings for parking, leading to soft ground story failure under earthquake. According to IS 1893 (2002) guidelines, these open ground stories should be designed for 2.5 times the design base shear obtained for the corresponding uniformly infilled frame building.

In addition to the above provisions, the design of an URM infilled RC building is governed by the provisions of Indian Standard IS 456 for the design of RC structures, and IS 13920 for the ductile detailing of buildings in seismic areas. In some cases, the criteria in these codes governs the size and reinforcement of the members and has been considered in the present study.

However, the codes are silent about modeling of infills in frame buildings and their effect on the overall behavior and seismic performance of buildings. Therefore, infills are generally ignored in design, assuming that they will provide additional strength and stiffness, which will result in improved performance.

4 Parametric study

This paper presents a parametric study on two sets of multi-story RC frame buildings with and without infills. The first set of buildings is designed for gravity loads only, while the second set is designed for gravity and earthquake loads as per relevant Indian Standards (IS 456, 2000; IS 875, Part 1 and Part 2, 1987a,b; IS 1893, 2002; IS 13920, 1993). In the later case, all the ductility related provisions of Indian Standard IS 1893 and IS 13920 for 'Special Moment Resistant Frames (SMRF)' are incorporated. The two sets of buildings are designated as 'gravity designed' and 'SMRF,' respectively. Note that Indian code designated SMRF buildings are different from SMRF buildings defined in ASCE 7 (2006). Indian SMRF buildings are actually equivalent to 'Intermediate Moment Resistant Frames (IMRF)' buildings of ASCE 7 in terms of ductile detailing and capacity design criteria, and are designed with a response reduction factor of 5. In Indian SMRF buildings, capacity design to ensure flexural yielding of beams prior to shear failure of beams and columns is ensured, but there is no provision to ensure strong column-weak beam design. In the case of infilled frame buildings, infills are considered to be uniformly distributed throughout the height of the buildings in the present study.

4.1 Design of generic buildings

The buildings considered in the parametric study have identical plan geometry as shown in Fig. 2 and have four stories. The plan is of an existing hospital building in New Delhi. It is symmetric in the longitudinal direction, and slightly asymmetric in the transverse

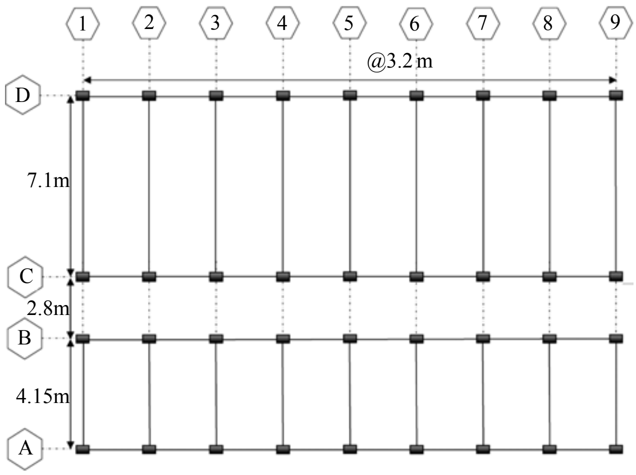

Fig. 2 Plan of buildings considered in the study

direction, and has significantly different redundancy in the two directions. Further, the spans of the beams in the two directions are also quite different, representing the characteristics of a wide range of real buildings. The story height has been considered as 3.3 m with foundations being 1.5 m below the ground level. The corridor is free from transverse beams, a typical feature of commercial and institutional buildings in India.

The buildings have been assumed to be situated on hard soil in seismic zone IV (Effective Peak Ground Acceleration, EPGA = 0.24 g for Maximum Considered Earthquake, MCE). M20 concrete and Fe 415 steel considered for the design and member sections have been proportioned to have about 2%–4% steel in the columns and about 1% steel (on each face) in the beams, wherever permitted by other code requirements. This results in square column sizes of 250 mm to 300 mm and beam sizes varying from 250 mm × 250 mm to 250 mm × 450 mm for the building designed for gravity loads alone, and for the building designed as SMRF, the section sizes vary from 250 mm × 250 mm to 375 mm × 375 mm for columns and 250 mm × 250 mm to 250 mm × 450 mm for beam members. The slab thickness has been assumed as 150 mm and a uniform weight of 0.5 kN/m^2 has been considered for flooring. The thickness of the infill has been considered as 115 mm and 230 mm for interior and exterior partitions, respectively, as per the prevailing practice in India. The weight of the infill is considered to be uniformly distributed along the length of the supporting beams.

The dead load and live load have been estimated using Indian Standard IS 875, Part-1 and Part-2 (1987a,b), respectively. Seismic design has been performed as per Indian Standard IS 1893 (2002), considering the specified load combinations. Preliminary sizes of beams have been calculated based on deflection criterion as per Indian Standard IS 456(2000). The minimum and maximum reinforcement criteria of IS 456 and IS 13920 have also been applied. For earthquake resistant designed buildings, the correction factor for the increased base shear due to capping on the design period (Eq. (1)) for the presence of URM infills has been applied.

4.2 Modeling of infills and their effect on seismic performance

Simulation of real behavior of infilled frames is a difficult task, as they exhibit complex nonlinear behavior due to infill-frame interaction. In the literature, two approaches are used for modeling of URM infills, generally known as micro-models and macro-models. Micro-models are based on continuum modeling of infills using finite element or discrete element simulation. These models are able to capture the behavior and interaction of infills with the frame in a very detailed manner, but they are computationally very expensive and are not suitable for use in a design office. In contrast, macro models are based on a physical understanding of the behavior of infill panel as a whole and are able to efficiently simulate the gross behavior of infills with sufficient accuracy. In this study, a macro model of infills as per ASCE 41 (2007) has been used for linear and nonlinear analyses.

Simulation of an infill panel requires estimation of effective stiffness and strength of equivalent diagonal strut. As per ASCE 41, the thickness and modulus of elasticity of the equivalent strut is considered to be the same as that of the infill, whereas the width of the equivalent strut is given as

$$a = 0.175\left(\lambda_1 h_{\mathrm{col}}\right)^{-0.4} r_{\mathrm{inf}} \tag{2}$$

where,

$$\lambda_1 = \left[\frac{E_{\mathrm{me}} t_{\mathrm{inf}} \sin 2\theta}{4 E_{\mathrm{fe}} I_{\mathrm{col}} h_{\mathrm{inf}}}\right]^{1/4}$$

h_{col} = column height between centerlines of beams
h_{inf} = height of infill panel
E_{fe} = expected modulus of elasticity of frame material (concrete)
E_{me} = expected modulus of elasticity of infill material
I_{col} = moment of inertia of column
L_{inf} = length of infill panel
r_{inf} = diagonal length of infill panel
t_{inf} = thickness of infill panel and equivalent strut

Infills have two prominent modes of failure. These may fail in shear during transfer of story shear across the infill panel confined within a bay of a frame. ASCE 41 provides the following relationship to calculate the expected in-plane shear strength of infill (R_{sh})

$$R_{\mathrm{sh}} = A_{\mathrm{ni}} f_{\mathrm{vie}} \tag{3}$$

where A_{ni} is the net mortared sectional area across the infill panel and f_{vie} is the expected bed-joint shear strength of the masonry in the infill.

Masonry infills experience diagonal compression and may fail due to either crushing or buckling, depending on the slenderness ratio. The strength of the infills in diagonal compression (ACI 530, 2005) can be expressed as

$$R_{\mathrm{dc}} = \frac{0.5 h_{\mathrm{inf}} t_{\mathrm{inf}} f_{\mathrm{a}}}{\cos\theta} \tag{4}$$

where θ is the inclination of the equivalent strut to the horizontal and f_{a} is the reduced compressive strength taking into account the slenderness effect.

ACI 530 (2005) provides a limit value of f_{a} depending on the slenderness ratio of masonry piers, which can be expressed as follows in the case of infill panels

$$f_{\mathrm{a}} = f'_{\mathrm{m}}\left(1 - \left(\frac{r_{\mathrm{inf}}}{140\lambda}\right)^2\right) \quad \text{if} \quad \frac{r_{\mathrm{inf}}}{\lambda} \le 99 \tag{5}$$

$$f_a = f'_m \left(\frac{70\lambda}{r_{inf}} \right)^2 \quad \text{if} \quad \frac{r_{inf}}{\lambda} > 99 \qquad (6)$$

where f'_m is the expected compressive strength of the infill masonry, and λ is the radius of gyration of the equivalent strut.

The minimum of the strengths in the two failure modes described above has been considered for computation of the strength of each strut element. The inelastic behavior of strut elements has been simulated by providing axial plastic hinges at mid-length of the struts. The force-deformation properties of equivalent struts have been derived using the ASCE 41 guidelines.

Figures 3 (a) and 3 (b) show typical load deformation curves of equivalent struts for a 115 mm thick partition and 230 mm thick peripheral infills, respectively. Sliding shear failure of masonry is the governing criteria of strength for most of the partition and peripheral infills. Whereas, diagonal compression failure of masonry governs the strength in case of longer infills (7.1 m long bays in Fig. 2), which have a slenderness ratio greater than 99 (Eq. (6)). The effect of infills on the fundamental periods of the two sets of buildings is shown in Table 1. The table also shows the design periods prescribed in IS 1893 (2002) to calculate the design base shear. As the code specified design period of a bare frame (T_a = 0.075 h, where h is height of the building in m) depends only on the height of the building, the design periods in the longitudinal and transverse directions are identical. However, for infilled frames, the design period (Eq. 1) depends on the plan dimension of the building in the considered direction, in addition to the height. Further, the code does not differentiate in different design levels for estimation of the design period and hence the design periods are identical for gravity designed and SMRF buildings. It can be observed from Table 1 that infills result in a drastic reduction in the fundamental periods of both gravity designed and SMRF buildings. Further, the code specified design periods are much smaller than the analytically obtained periods for bare and infilled frames. Therefore, the capping on the design period results in a significant increase in the design base shear.

The seismic performance of different buildings has been estimated using nonlinear static pushover analysis as per ASCE 41. The analysis has been carried out using nonlinear analysis software SAP 2000 Nonlinear (2010). In the analytical model, flexural (M) hinges are assigned at both ends of the beams, whereas axial force-moment interaction (P-M-M) hinges are assigned to columns. Axial plastic hinges as described earlier have been assigned at the mid-length of the equivalent diagonal struts to simulate the infills. Figures 4 and 5 compare the seismic performance of gravity designed and SMRF buildings, respectively, with and without

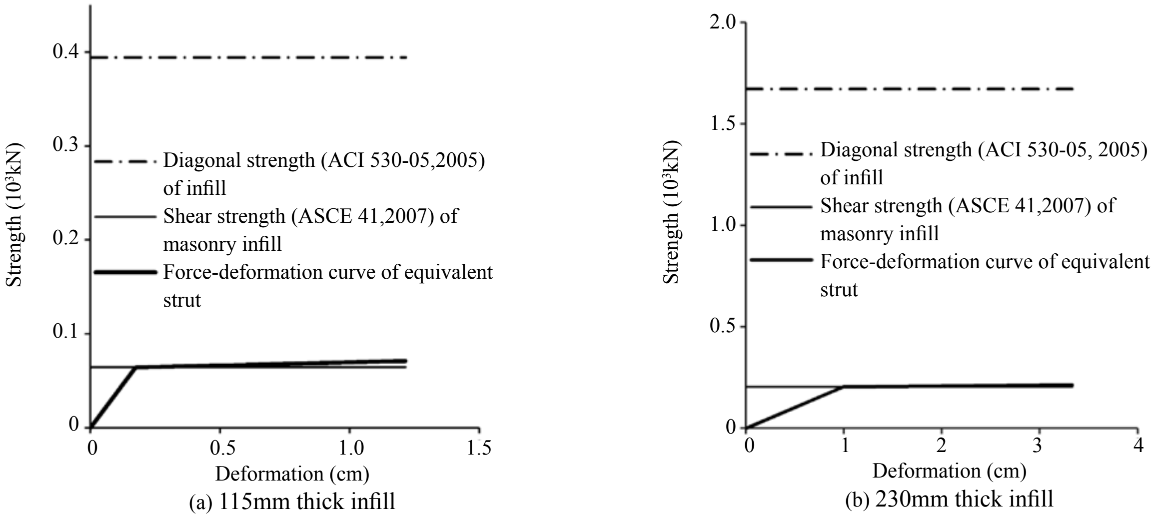

Fig. 3 Typical load deformation curve for equivalent strut

Table 1 Effect of infills on period of 'gravity designed' and 'SMRF' RC buildings

Design level	Frame configuration	Fundamental period (from analysis) (s)		Design period (from IS: 1893, 2002) (s)	
		Longitudinal	Transverse	Longitudinal	Transverse
Gravity designed	Bare	1.750	1.971	0.563	0.563
	Infilled	0.403	0.498	0.261	0.353
SMRF	Bare	1.317	1.52	0.563	0.563
	Infilled	0.384	0.458	0.261	0.353

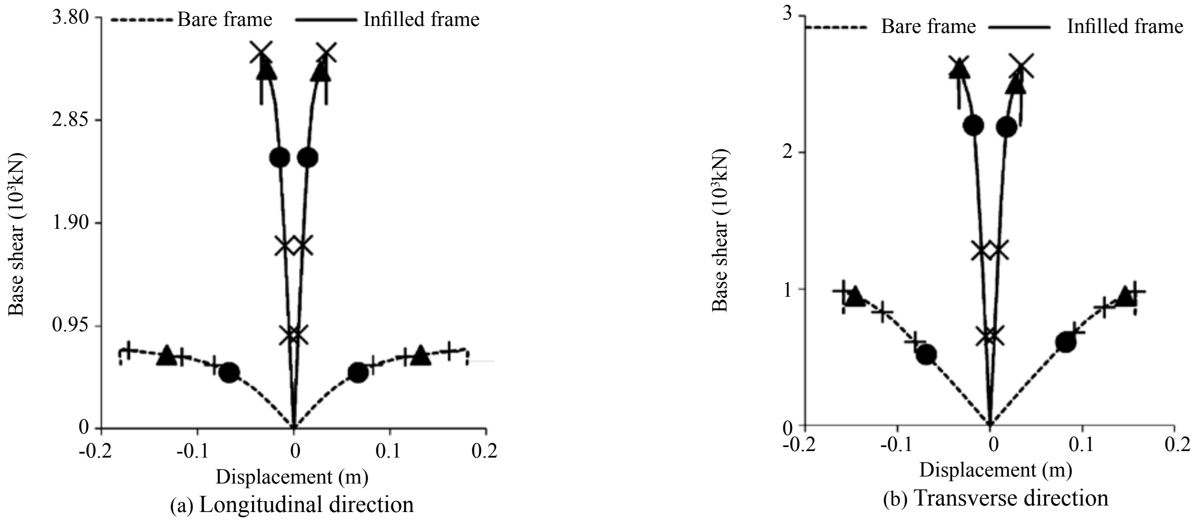

Fig. 4 Comparison of capacity curves and performance points for bare frame and uniformly infilled frame buildings designed for gravity load as per relevant Indian Standards; the black dot (●) represents the performance point for Design Basis Earthquake and black triangle (▲) represents the performance point for Maximum Considered Earthquake; the three crosses (×) represent 'Immediate Occupancy', 'Life Safety', and 'Collapse Prevention' performance levels, consecutively

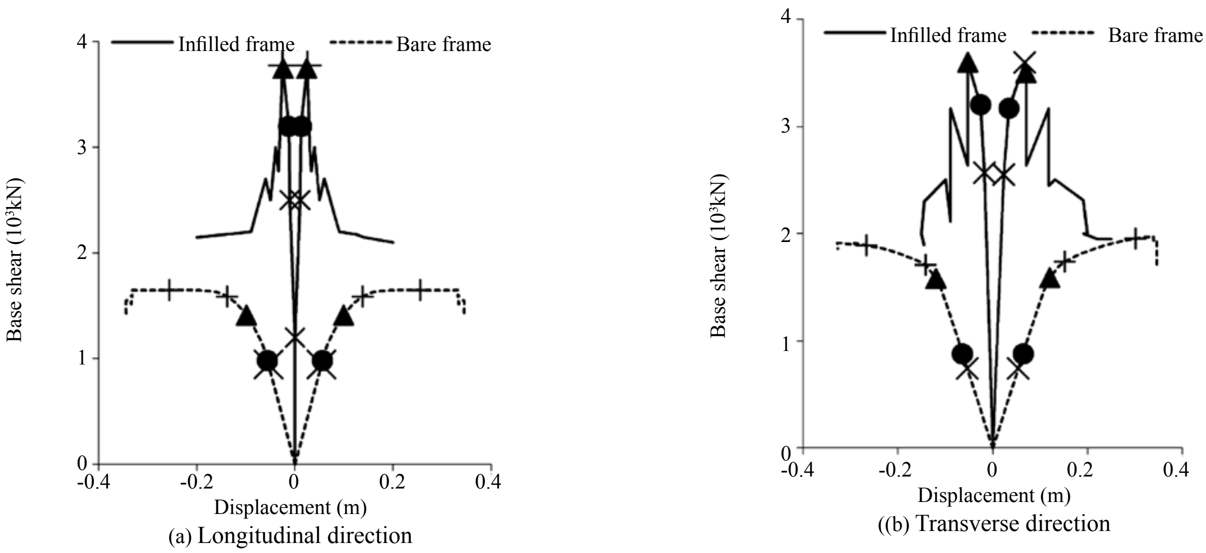

Fig. 5 Comparison of capacity curves and performance points for bare frame and uniformly infilled frame buildings designed as SMRF as per relevant Indian Standards; the black dot (●) represents the performance point for Design Basis Earthquake and black triangle (▲) represents the performance point for Maximum Considered Earthquake; the three crosses (×) represent 'Immediate Occupancy', 'Life Safety', and 'Collapse Prevention' performance levels, consecutively

infills. The figures also show the performance points of the buildings for the Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) of seismic zone IV. The performance points have been obtained using the Displacement Modification Method (DMM) of FEMA 440(2006b), and the performance levels (Immediate Occupancy, Life Safety and Collapse Prevention) shown in the figures correspond to the acceptance criteria of ASCE 41 (2007) for infills and RC members. Note that ASCE 41 specifies performance levels in terms of plastic rotations for RC beams and columns and in terms of drift ratio for infills. The performance levels have been marked on the building pushover curve by identifying the pushover step, where the first member (beam, column, or infill) in the building undergoes the plastic rotation, as specified in ASCE 41 for the respective performance level. With a sufficiently large number of analysis steps, the performance levels can be marked with acceptable accuracy.

It can be observed from the capacity curves shown in Figs. 4 and 5 that the stiffness and strength of both the gravity designed and SMRF buildings increase greatly due to the infills, but there is a drastic reduction in ductility. The reduction in ductility is primarily due to failure of the infills at much lower lateral displacement. In the gravity load designed building, the RC frame members also fail much earlier as compared to the bare frame, resulting in collapse of the building; whereas in the case of the SMRF building, the RC frame members continue to resist the lateral load, even after failure of the infills. The software used in the present study is not able to continue the analysis further after failure of a large number of infills. Therefore, to obtain the capacity curve after failure of infills, the failed infills were removed from the model and the revised model was re-analyzed. Further, in the SMRF design, the strength and ductility capacities increase significantly, both for the bare frame as well as for the infilled frame. It is also interesting to note that even the building designed without any consideration for earthquake forces (i.e., gravity designed building), is able to sustain a MCE of zone IV without collapse (Fig. 4) both with a bare frame and an infilled frame. The overall effect of URM infills is deterioration of the seismic performance of the buildings, which is more prominent for the SMRF (Fig. 5). In this case, the performance of the bare frame building is Immediate Occupancy and Life Safety for DBE and MCE, respectively, but it deteriorates to Collapse Prevention with the URM infills. Nonetheless, all the studied buildings survive the MCE of zone IV without collapse. However, these observations are based on deterministic analysis for average (median) buildings of the considered categories. It will be interesting to study the effect of various uncertainties in capacity and demand parameters on the expected performance, presented in the next section on fragility analysis.

5 Fragility analysis

Seismic vulnerability (or fragility) of a structure is described as its susceptibility to damage by the ground shaking of a given intensity. It is expressed as a relationship between the ground motion severity (i.e., intensity, PGA, or spectral displacement) and structural damage (expressed in terms of damage grades). A number of approaches are available (Calvi *et al*., 2006) for developing the vulnerability relations for different types of buildings, ranging from those based on the empirical damage data from the past earthquakes to those based on purely analytical simulations.

This study uses HAZUS methodology (2003, 2006a) to develop fragility curves for the considered buildings with and without URM infills. The methodology was originally developed for seismic risk assessment in the USA but has been extensively used throughout the world. In HAZUS methodology, the fragility curves are lognormal distributions that represent the probability of being in or exceeding a given damage state, given as

$$P[\mathrm{ds}/S_d] = \Phi\left[\frac{1}{\beta_{\mathrm{ds}}}\ln\left(\frac{S_d}{\overline{S}_{d,\mathrm{ds}}}\right)\right] \quad (7)$$

where $\overline{S}_{d,\mathrm{ds}}$ is the median spectral displacement for damage state ds, Φ is a normal cumulative distribution function, and β_{ds} is the standard deviation of the natural logarithm of the spectral displacement for damage state ds, which describes the combined variability, given as

$$\beta_{\mathrm{ds}} = \left\{\left(\mathrm{CONV}\left[\beta_{\mathrm{C}}, \beta_{\mathrm{D}}, \overline{S}_{d,\mathrm{ds}}\right]\right)^2 + \left(\beta_{\mathrm{M(ds)}}\right)^2\right\}^{(1/2)} \quad (8)$$

where β_{C} is the lognormal standard deviation parameter representing variability in the capacity properties of the building, β_{D} represents the variability in the demand spectrum due to spatial variability of the ground motion, and $\beta_{\mathrm{M(ds)}}$ represents the uncertainty in the estimation of the damage state threshold.

5.1 Damage state definition

An important step in developing the fragility functions is definition of various damage states. In Intensity Scales, these damage states are defined in descriptive terms, but for fragility analysis, they need to be defined in terms of engineering parameters. HAZUS (2003, 2006a) has used a two criteria approach, based on the performance levels of individual members, to define the damage state thresholds. Barbat *et al*. (2006) have proposed a simpler approach (Table 2) based on yield and ultimate spectral displacement of the buildings, and the same has been used in the present study. The yield spectral displacement (S_{dy}) and ultimate spectral displacement ($S_{d\mathrm{u}}$) are obtained analytically from the bi-linearization of capacity curves. The yield spectral displacement represents the point where a sizable number of members have yielded, resulting in significant loss of stiffness of the whole structure. Similarly, ultimate spectral displacement represents the point where the strength of the building degrades below 80% of the peak.

Table 2 Damage state definition (Barbat *et al*., 2006)

Damage grade	Damage state	Spectral displacement
Gr1	Slight damage	$0.7S_{dy}$
Gr2	Moderate damage	S_{dy}
Gr3	Extensive damage	$S_{dy}+0.25(S_{d\mathrm{u}}-S_{dy})$
Gr4	Complete damage	$S_{d\mathrm{u}}$

Table 3 shows the median spectral displacements corresponding to different damage grades, obtained from the capacity curves shown in Figs. 4 and 5, converted into Acceleration Displacement Response Spectrum (ADRS) format as per the ATC 40 (1996) procedure and using the criteria of Table 2.

5.2 Consideration of variability

Estimation of variability in fragility analysis is a complex process requiring a large amount of statistical

Table 3 Median spectral displacement corresponding to different damage grades of buildings with different design levels

Design level	Median S_d (mm)			
	Damage grade, Gr1	Damage grade, Gr2	Damage grade, Gr3	Damage grade, Gr4
Gravity designed bare frame	44	63	79	129
Gravity designed infilled frame	8	11	13	22
SMRF bare frame	58	83	110	189
SMRF infilled frame	8	12	17	31

data. HAZUS (2003) has presented variability for fragility estimation of American (Californian) buildings, where the total variability in structural damage is considered to be contributed by the three sources as described in Eq. (8) and is obtained by combining the three variabilities using a complex convolution process. Although India has suffered several major earthquakes in the past, unfortunately, such systematic data is lacking for Indian conditions. However, the aim of the present study is not to prescribe standard fragility functions for Indian buildings, but to examine the role of URM infills on the fragility of RC frame buildings. Therefore, the HAZUS values of variability, for the corresponding cases, as reproduced in Table 4, have been considered.

HAZUS has considered uniform variability (lognormal standard deviation parameter) in the demand spectrum as 0.45 and 0.5, for acceleration and velocity sensitive ranges of the spectrum. Similarly, moderate variability of 0.4 in the damage state threshold and 0.3 in the capacity curve for all structural damage states, as suggested by HAZUS, has been considered for all the buildings in the present study. As gravity designed buildings are constructed without any considerations of earthquake resistant measures or ductile detailing, these are expected to experience major degradation after yielding. Accordingly, a variability of 0.5 corresponding to major post yield degradation has been considered. In contrast, buildings designed and detailed as SMRF are expected to experience minor degradation after yielding. Therefore, variability of 0.9 corresponding to minor post yield degradation has been considered for SMRF. However, the presence of infills results in rapid post yield degradation of these buildings and the same has been accounted for using increased variability as shown in Table 4.

Table 4 Variability parameters considered for gravity designed and SMRF buildings (as per HAZUS, 2003)

Building design levels	Damage state	Post-yield degradation	Damage state variability ($\beta_{M(ds)}$)	Capacity curve variability (β_c)	Total variability (β_{ds})
Gravity designed bare frame	Gr3	Major degradation (0.5)	Moderate (0.4)	Moderate (0.3)	0.85
	Gr4				
Gravity designed infilled frame	Gr3	Major degradation (0.5)			0.85
	Gr4	Extreme degradation (0.1)			1.00
SMRF bare frame	Gr3	Minor degradation (0.9)			0.75
	Gr4	Major degradation (0.5)			0.85
SMRF infilled frame	Gr3	Minor degradation (0.9)			0.75
	Gr4	Major degradation (0.5)			0.85

5.3 Fragility curves and DPMs

Fragility of bare frame and uniformly infilled frame buildings has been compared to study the effect of URM infills on the fragility of RC frame buildings. Figs. 6 and 7 show the fragility curves of bare and infilled frames for the two considered design levels.

The fragility curves indicate much higher damage probabilities of infilled frames as compared to bare frames for gravity-designed buildings as well as for SMRF buildings. However, it should be noted that fragility curves compare the probability of damage for a given

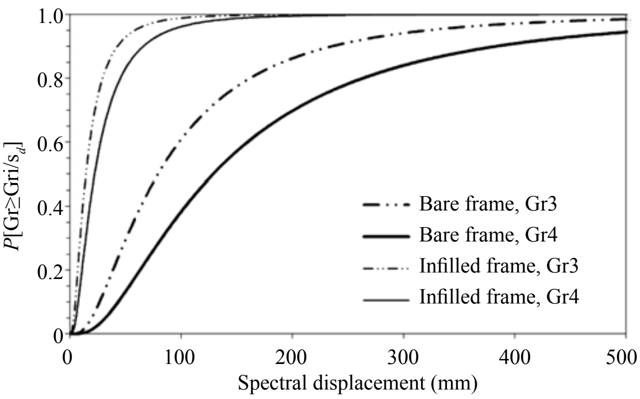

Fig. 6 Comparison of fragility for damage grades Gr3 and Gr4 of bare frame and uniformly infilled frame buildings designed for gravity load only

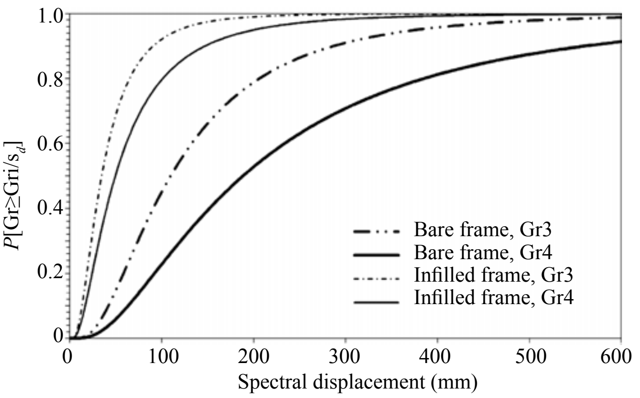

Fig. 7 Comparison of fragility for damage grades Gr3 and Gr4 of bare frame and uniformly infilled frame buildings designed as SMRF

spectral displacement, but as the dynamic characteristics of infilled buildings are significantly different from those of bare frames, the damage probabilities cannot be directly compared from Figs. 6 and 7. For the purpose of comparison, the damage probabilities need to be expressed in terms of Peak Ground Acceleration (PGA). Spectral shape and hysteretic damping plays an important role in the relationship between PGA and spectral displacement. In the present study, the spectral shape of IS 1893 (2002) for Soil Type-I has been used in the Displacement Modification Method (DMM) of ASCE 41 to obtain the spectral displacement corresponding to different values of PGA. Table 5 shows the damage probabilities for PGA corresponding to DBE (0.12 g, 0.18 g) and MCE (0.24 g, 0.36 g) of Indian seismic zones IV and V, respectively.

It is interesting to note that in the deterministic performance analysis presented in the previous section, the buildings which survived the MCE of zone IV (PGA = 0.24g) without collapse, show more than 50% probability of complete damage (except for the SMRF bare frame, which has a 24% chance of complete damage) in the fragility analysis. This demonstrates the effect of inherent variabilities on the expected performance of buildings. Again, the gravity designed infilled frame buildings show the worst performance and have the highest damage probability for all values of PGA. It can also be observed from Table 5 that inclusion of infills increases the damage probability of the frame buildings significantly, irrespective of the design level. Considering this adverse effect of infills on the performance of frame buildings, HAZUS (2003, 2006a) does not consider frame buildings with URM infills as moderate or high code design.

Table 5 Damage probabilities for bare frame and infilled frame buildings %

Building	Damage ≥Gr3 PGA (g)				Damage ≥Gr4 PGA (g)			
	0.12	0.18	0.24	0.36	0.12	0.18	0.24	0.36
Gravity designed bare frame	40	59	71	85	21	37	50	68
Gravity designed infilled frame	60	76	86	94	38	54	65	79
SMRF bare frame	16	32	47	68	6	15	24	41
SMRF infilled frame	45	66	79	91	20	36	50	68

6 Conclusion

The dynamic characteristics and seismic performance of frame buildings are significantly affected by the inclusion of URM infills. The period of vibration of frame buildings is substantially reduced by the presence of these infills, while the performance of both gravity designed and SMRF buildings significantly deteriorate. The fragility curves indicate higher damage probability of infilled frames as compared to bare frames for both gravity designed buildings and SMRF buildings designed per relevant Indian codes. Gravity designed infilled frame buildings have the worst performance and highest damage probability for all grades of damage. The inclusion of infills significantly increases the damage probability of the frame buildings irrespective of design level. Even the SMRF buildings designed and detailed as per Indian codes have a 50% probability of complete damage under MCE in the same seismic zone for which they were designed. Considering the significant undesirable effect of URM infills on seismic performance, it is very important to give proper attention to the infill-frame interaction in the design of URM infilled RC frame buildings. Further, this study shows that a deterministic performance analysis does not provide complete insight into the expected performance of buildings, and a probabilistic framework for performance-based design is required.

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