INFLUENCE OF MASONRY INFILL ON THE COLLAPSE PERFORMANCE OF RC FRAME BUILDINGS

A. Jain[(1)], M. Kumar[(2)], H. Sarma[(3)]

[(1)] Graduate student, Indian Institute of Technology Gandhinagar, India, ankushjain793@gmail.com
[(2)] Assistant Professor, Indian Institute of Technology Gandhinagar, India, mkumar@iitgn.ac.in
[(3)] PhD Candidate, Florida International University, USA (formerly, Junior Research Fellow at Indian Institute of Technology Gandhinagar, India), hsarm004@fiu.edu

Abstract

Unreinforced masonry-infilled reinforced concrete (RC) frames are widely seen in many parts of the world. The lateral force-displacement response of the infilled RC frames is rather complex due to multiplicity of possible brittle failure modes for the masonry panel and the interaction between frame and infills, which explains the absence of comprehensive guidelines from most design standards across the world. Most design codes treat masonry infills in the RC frame structures as non-structural elements. Some codes do consider the infills as structural elements, but do not recommend accounting for the strength and stiffness characteristics of masonry infill at the design stage. This paper presents a study on the influence of infill on the performance of infilled frames during extreme earthquakes. A total of 12 frames are considered that differ in seismic zone according to Indian earthquake code (III or V), number of stories (three or nine), presence of infill (bare frames or uniformly infilled frames), and masonry prism strength (3 MPa or 9 MPa). Bare frames are designed first considering the dead weight of masonry. Infills are introduced in the frames thus designed. A recently proposed macro model is used to model the frames. The model is validated against reported experimental studies on single-bay-multi-story infilled RC frames. Pushover analyses on the frames suggest that infills contribute to significant increase in lateral strength and stiffness of the frames, and that the ground story columns in an infilled frame attract shear forces much greater than respective design shear forces. Collapse margin ratio (CMR) was calculated in accordance with the provisions of FEMA P695. This ratio indicates the factor by which the maximum considered earthquake (MCE) spectrum would be increased to achieve a 50% probability of collapse. Increase in masonry prism strength from 3 MPa to 9 MPa led to an increase in the CMR by 60 – 190%. The CMR was considerably lower for frames in seismic zone V (higher seismic zone) compared to zone III, and for nine-story frames compared to three-story frames. The reduction in the CMR for a higher seismic zone was more pronounced for a greater masonry prism strength. Overall, presence of a "strong" infill was found beneficial for collapse performance of infilled RC frames, but it also meant that columns would attract much greater forces compared to their design forces making them vulnerable to shear failure. Noting that strength of infill is not accounted for at the stage of design of frame buildings, masonry can induce great amount of uncertainty in the performance of infilled RC frames and its individual components.

Keywords: Masonry infill; RC frames; Pushover analysis; Incremental dynamic analysis; Collapse margin ratio;

1. Introduction

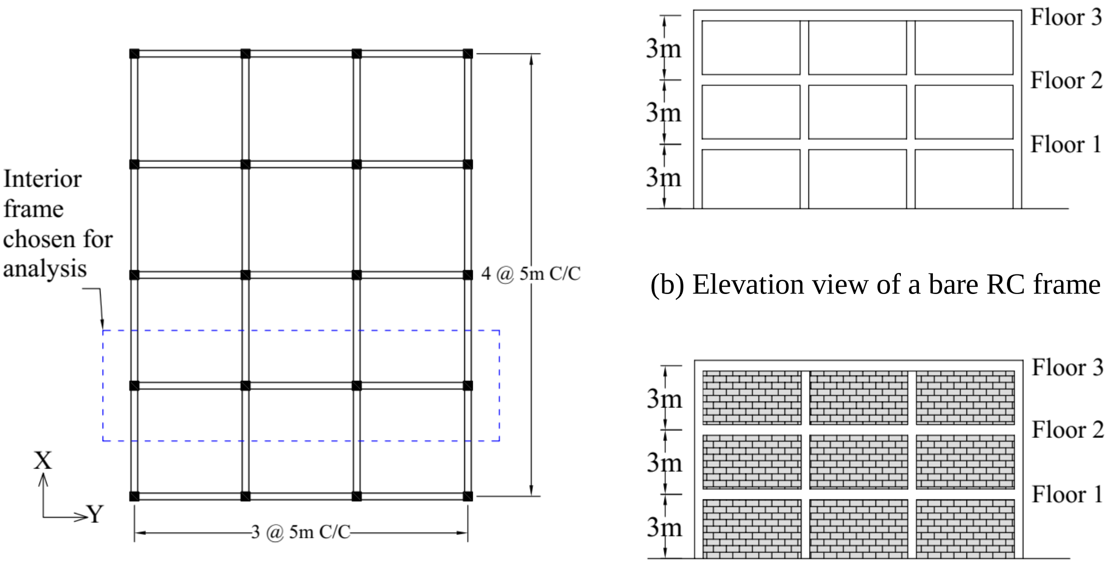
Unreinforced masonry panels are used in reinforced concrete (RC) frame buildings for functional and aesthetic reasons. Physical properties of the frame and the panels are considerably different. For example, a masonry panel can be 10 – 20 times stiffer compared to the surrounding frame (e.g., [1]). A frame can be designed to behave in a ductile manner, while an unreinforced masonry panel can fail through a wide spectrum of a brittle failure modes (e.g., [2]). The response of an RC frame structure with masonry infill is often quite complex as a result, and most design codes across the world ignore the contribution of infill panel (e.g., [3]). Many codes that consider the masonry infill panel explicitly permit the distribution of design forces between frame and infill in a manner that can lead to a non-ductile global response (e.g., [1]). Some other codes put a threshold on the minimum design for the frame explicitly or implicitly. None of these codes consider the strength of infill in the design of the frame to the knowledge of the authors. As noted in the past (e.g., [1], [5]), a stronger infill can be detrimental to seismic performance. This paper presents a study on the influence of strength of masonry panel on the collapse capacity of RC frame buildings conforming to a design standard.

Performance of infilled RC frames under extreme loading conditions can be assessed through the methodology presented in FEMA P695 [4]. Sattar and Liel [6] investigated the collapse performance of four- and eight-storied non-ductile RC frames with and without masonry infills per the FEMA procedure. Frame members were modeled with plasticity lumped at the ends of the elastic elements. Masonry was modeled using a single strut. Shaking intensity corresponding to collapse for uniformly infilled frame was found greater compared to partially infilled or bare frame. Sattar and Liel [5] considered two-, four- and eight-storied non-ductile RC frame buildings with and without masonry infills. Two types of infills were considered: “strong” and “weak.” Frame members were modeled in a manner similar to Sattar and Liel [6]. Masonry panel was replaced with a pair of off-diagonal struts with the constitutive model defined based on a detailed finite element analysis. Infilled RC frames with “strong” infills were found more vulnerable to collapse compared to bare frames or frames with “weak” infills. Pisode et al. [7] studied four- and eight-storied RC frame buildings with uniformly placed infills, with infills placed uniformly everywhere except in the ground story, and with no infill. Ductile as well as non-ductile frames were considered. Non-ductile frames were found more vulnerable to collapse compared to ductile frames. Burton and Sharma [8] studied six- and nine-storied frames with masonry infills present uniformly across the building and absent only in the ground story. The buildings were subjected to sequences of main- and after-shocks. Global as well as local damage measures were considered to characterize collapse in the buildings. Frame was modeled using elastic elements with plastic behavior concentrated at the ends of the elements. Masonry was replaced with a diagonal and an off-diagonal struts. Buildings with open ground story were found more vulnerable to change in mainshock intensity. Through a procedure different from FEMA P695, Suthasit and Warnitchai [9] studied seismic performance of ductile RC frames with number of stories ranging between two and six, with no infill, and with uniformly placed infills having a range masonry prism strength. Masonry was replaced with a pair of off-diagonal struts in the model. The shear force in the column for the infilled frame was compared with design shear capacity to assess local shear failure. Shorter frames with stiffer infills were found to have a smaller probability of exceeding maximum considered earthquake drift limit.

This paper presents a study on three- and nine- story RC frames with infills placed uniformly and with no infill in two seismic zones of India. Two values of masonry prism strength are considered: 3 MPa and 9 MPa. The macro model for infilled RC frames proposed by Ghosh and Kumar [10] is used. This model is capable of incorporating shear and flexural failure in frame members, and interaction between the axial load and moment capacity. Masonry is represented using a diagonal and two off-diagonal struts, constitutive model for which was developed by modifying the models proposed in the past [10] (also see [11]). The model was shown to capture the initial stiffness, peak strength and post-peak response for a large number of experimentally studied one-bay-one-story infilled RC frames reasonably well. Pushover analyses are carried out to understand the lateral force-displacement response of the frames, and the contributions of the infills. A series of incremental dynamic analyses are performed to determine the drift and intensity parameters associated with collapse in line with the procedure presented in FEMA P695 [4]. These results provide insights into the influence of masonry infills on the collapse capacity of frames designed considering masonry as a non-structural element.

2. Description of buildings

Buildings considered in the present study have a plan dimension of 20 m × 15 m, and have four and three bays along X and Y direction, respectively. Length of each bay is 5 m center-to-center. Buildings have three or nine stories. Height of a story in a building is 3 m. Thickness of the floor slab is 125 mm. Fig. 1(a) presents the plan view of the building. Also identified in the panel is the frame considered for further study. Panel (b) of Fig. 1 shows the elevation view of the frame in a three-storied building. Panel (c) presents the frame with infills placed uniformly.



(a) Typical floor plan (c) Elevation view of a RC frame with infill

Fig. 1 – Plan and elevation views of a representative building

An imposed load of 2 kN/m^2 is considered on the floors of building in accordance with IS 875, Part 2 [12]. Masonry walls are considered 230 mm thick, wherever applicable. Two values of masonry prism strength are considered: 3 MPa and 9 MPa. Buildings are located in seismic zones III and V according to Indian earthquake code IS 1893, Part 1 [13]. Seismic zone factors for the two zones are 0.16 and 0.36, respectively. The zone factors correspond to the peak ground accelerations (PGAs) during maximum considered earthquake (MCE). Design base shear (V_B) is calculated considering importance factor (I) for the buildings as 1.0, response reduction factor (R) of 5, and soil type to be medium. Table 1 presents the general details of a total of 12 frames considered in this study. Dead weight of masonry was considered for all frames including those marked 'bare' in Table 1. Characteristic strength of concrete f_{ck} is also shown in the table. Yield strength of main and transverse steel reinforcements (f_y) are taken as 500 MPa for all frames. Table 2 presents a summary of beam and column cross-sections. Additional details on the design of frames are presented in Jain [14].

3. Analytical model

Open source software platform OpenSees PEER [15] is used to model the frames identified in Table 1. As noted previously, macro model proposed by Ghosh and Kumar [10] is used to model the frames. Frame members are modeled using a distributed plasticity element *forceBeamColumn*. This element is capable of capturing nonlinear behavior in flexure and the interaction between axial load and bending moment. The constitutive behavior for concrete is defined using the model proposed by Mander et al. [16]. *Concrete01* material available with OpenSees is used to model the core and cover concrete. Longitudinal reinforcement is modeled using *Steel01* element, with the post-yield slope of material stress-strain curve set equal to 0.1% of the elastic modulus. Zero-length shear hinge elements are introduced in the frame members at multiple locations. These elements are defined using *Pinching4* material. Cyclic degradation and pinching parameters are defined based on Kumar et al. [1]. Shear behavior of frame members is defined using the model proposed by Elwood and Moehle [17], wherein peak shear strength is defined according to Sezen and Moehle [18]. Infill is modeled using a set of three struts: one diagonal and two off-diagonal. Strut width proposed by Mainstone [19] is considered.

Table 1 – General details on RC frames considered in this study

No. of Story	R	Zone	Infilled /Bare[1]	Prism strength, f_m' (MPa)	Frame ID	f_{ck} (MPa)
3	5	III	Bare	-	S3-Z3-Bare	25
			Infilled	3	S3-Z3-3MPa	25
			Infilled	9	S3-Z3-9MPa	25
		V	Bare	-	S3-Z5-Bare	25
			Infilled	3	S3-Z5-3MPa	25
			Infilled	9	S3-Z5-9MPa	25
9	5	III	Bare	-	S9-Z3-Bare	30
			Infilled	3	S9-Z3-3MPa	30
			Infilled	9	S9-Z3-9MPa	30
		V	Bare	-	S9-Z5-Bare	40
			Infilled	3	S9-Z5-3MPa	40
			Infilled	9	S9-Z5-9MPa	40

[1] Bare frame is designed considering the weight of masonry. Mass of masonry is included in the dynamic analysis of bare and infilled frames. Struts are used to simulate the strength and stiffness properties of masonry for infilled frames.

Constitutive relationship proposed by Ghosh and Kumar [10] was developed by modifying existing constitutive models. The model was demonstrated to capture the global and local responses of one-bay-one-story infilled RC frames well for a large number of experimentally studied specimens. Elements representing struts were defined using *Pinching4* material, and corresponding cyclic degradation and pinching parameters were defined based on Kumar et al. [1]. Fig. 2(a) presents a schematic of a uniformly infilled three-story-three-bay frame with some beams, columns and masonry panels replaced with elements representing the three components. Fig. 2(b) presents the schematic of the analytical model for a one-bay-one-story infilled RC frame. Solid (broken) lines representing the masonry panel will be in compression (tension) when a force is applied at the beam towards the rightward (leftward) direction. Tensile strength and stiffness of the elements representing the masonry panel are set equal to zero.

A gravity load of 1.05 times the dead load and 0.25 times the live load is applied at every level of a frame in accordance with the FEMA P695 [4] procedure. Masses are lumped at the beam-column joints. A Rayleigh damping of 5% is defined with the definition of damping anchored to the first mode and n^{th} mode of a frame, where n is the number of stories in a frame.

4. Validation of analytical model

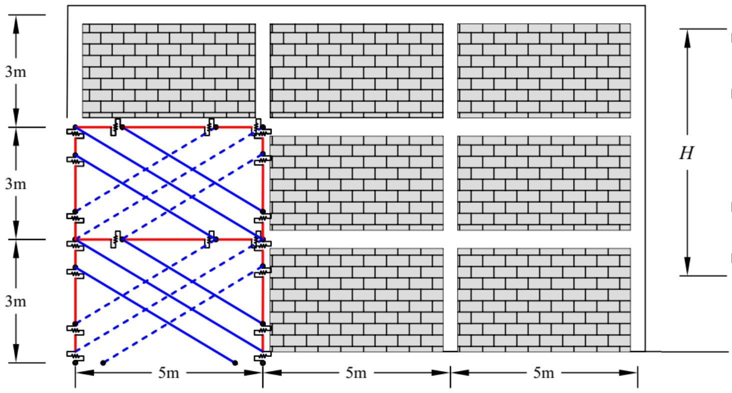
Suzuki et al. [20] had performed an experimental study on a two-story-one-bay masonry-infilled RC frame (specimen 2S-1B). The specimen was modeled using the approach presented in Section 3. Details on the experiments and modeling are presented in Jain [14]. The backbone of experimentally obtained lateral force-displacement response is presented in Fig. 3(a). Also presented in the figure is the pushover curve for the specimen obtained through the analysis. Koutas et al. [21] tested a three-story-one-bay masonry-infilled RC frame. Ductile design provisions were not followed in the design of the frame. Details on the experiment and modeling can be seen in Jain [14]. Experimentally obtained backbone curve for the lateral force-displacement response and analytically obtained pushover curve are presented in Fig. 3(b). Analytical models are able to capture the experimental observations reasonably well.

Table 2 – Details of beam and column cross-sections

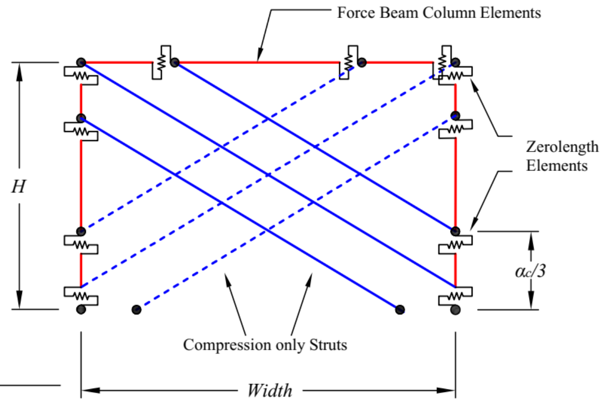
	Column			**Beam**			
	Cross-section (mm)	**Main reinf.**	**Traverse reinf. at column ends**	**Cross-section (mm)**	**Main reinf. at top near support**	**Main reinf. at bottom near support**	**Transverse reinf. near support**
S3-Z3							
Story 1	400 × 400	8# 16φ	4-Leg 8φ @ 75	300 × 500	3# 16φ	3# 12φ	2-Leg 8φ @ 75
Story 2	400 × 400	8# 16φ	4-Leg 8φ @ 75	300 × 500	3# 16φ	3# 12φ	2-Leg 8φ @ 75
Story 3	400 × 400	8# 16φ	4-Leg 8φ @ 75	300 × 500	3# 16φ	3# 12φ	2-Leg 8φ @ 75
S3-Z5							
Story 1	400 × 400	16# 20φ	4-Leg 8φ @ 75	300 × 500	5# 16φ	5# 12φ	2-Leg 8φ @ 75
Story 2	400 × 400	12# 20φ	4-Leg 8φ @ 75	300 × 500	5# 16φ	5# 12φ	2-Leg 8φ @ 75
Story 3	400 × 400	12# 12φ	4-Leg 8φ @ 75	300 × 500	3# 12φ	3# 12φ	2-Leg 8φ @ 75
S9-Z3							
Story 1	500 × 500	16# 20φ	4-Leg 8φ @ 75	300 × 500	6# 16φ	4# 16φ	2-Leg 8φ @ 75
Story 2	500 × 500	16# 16φ	4-Leg 8φ @ 75	300 × 500	6# 16φ	4# 16φ	2-Leg 8φ @ 75
Story 3	500 × 500	16# 16φ	4-Leg 8φ @ 75	300 × 500	6# 16φ	4# 16φ	2-Leg 8φ @ 75
Story 4	450 × 450	12# 16φ	4-Leg 8φ @ 75	300 × 500	6# 16φ	4# 16φ	2-Leg 8φ @ 75
Story 5	450 × 450	12# 16φ	4-Leg 8φ @ 75	300 × 500	5# 16φ	3# 16φ	2-Leg 8φ @ 75
Story 6	450 × 450	12# 16φ	4-Leg 8φ @ 75	300 × 500	5# 16φ	3# 16φ	2-Leg 8φ @ 76
Story 7	450 × 450	12# 16φ	4-Leg 8φ @ 75	300 × 500	5# 16φ	3# 16φ	2-Leg 8φ @ 77
Story 8	400 × 400	12# 16φ	4-Leg 8φ @ 75	300 × 500	4# 16φ	3# 16φ	2-Leg 8φ @ 78
Story 9	400 × 400	12# 12φ	4-Leg 8φ @ 75	300 × 500	4# 12φ	4# 12φ	2-Leg 8φ @ 75
S9-Z3							
Story 1	550 × 550	20# 25φ	4-Leg 8φ @ 75	300 × 500	5# 25φ	3# 25φ	2-Leg 10φ @ 75
Story 2	550 × 550	20# 25φ	4-Leg 8φ @ 75	300 × 500	5# 25φ	3# 25φ	2-Leg 10φ @ 75
Story 3	500 × 500	20# 25φ	4-Leg 8φ @ 75	300 × 500	5# 25φ	3# 25φ	2-Leg 10φ @ 75
Story 4	500 × 500	20# 20φ	4-Leg 8φ @ 75	300 × 500	5# 25φ	3# 25φ	2-Leg 10φ @ 75
Story 5	500 × 500	20# 20φ	4-Leg 8φ @ 75	300 × 500	5# 25φ	3# 25φ	2-Leg 10φ @ 75
Story 6	450 × 450	20# 20φ	4-Leg 8φ @ 75	300 × 500	6# 20φ	4# 20φ	2-Leg 10φ @ 75
Story 7	450 × 450	20# 20φ	4-Leg 8φ @ 75	300 × 500	6# 20φ	4# 20φ	2-Leg 10φ @ 75
Story 8	400 × 400	20# 16φ	4-Leg 8φ @ 75	300 × 500	4# 20φ	3# 20φ	2-Leg 10φ @ 75
Story 9	400 × 400	20# 12φ	4-Leg 8φ @ 75	300 × 500	4# 12φ	4# 12φ	2-Leg 10φ @ 75

5. Pushover analyses

Pushover analyses were carried out for all twelve frames of Table 1. Height-wise distribution of lateral load recommended by FEMA P695 [4] was considered, which corresponds to the fundamental mode shape of the frame at the beginning of the loading. Panel (a) of Fig. 4 presents the lateral force-displacement response of frame S3-Z3-Bare (see Table 1). Results for frames S3-Z3-3MPa and S3-Z3-9MPa are also presented in the figure. Panels (b), (c) and (d) of Fig. 4 present the results for remaining frames identified in Table 1. Presence of an infill (represented through struts in the model) substantially increases the initial lateral stiffness of the system. As an example, the initial stiffness of frame S3-Z3-Bare is 12,495 kN/m, which increases to 288,722 kN/m (643,216 kN/m) when masonry infill with a prism strength of 3 MPa (9 MPa) is present. Peak strength of a frame depends significantly on whether infill is modeled explicitly, and on the prism strength of the infill. As an example, the peak strength of frames S3-Z3-Bare, S3-Z3-3MPa and S3-Z3-9MPa are 433 kN, 1,543 kN and 3,137 kN, respectively. Peak lateral forces in infilled frames and in the struts representing masonry panels in the ground story are achieved simultaneously. Reduction in the lateral force in the infilled frame was sharper after peak force was achieved compared to bare frame; there was no clear trend with respect to increase in the masonry prism strength.

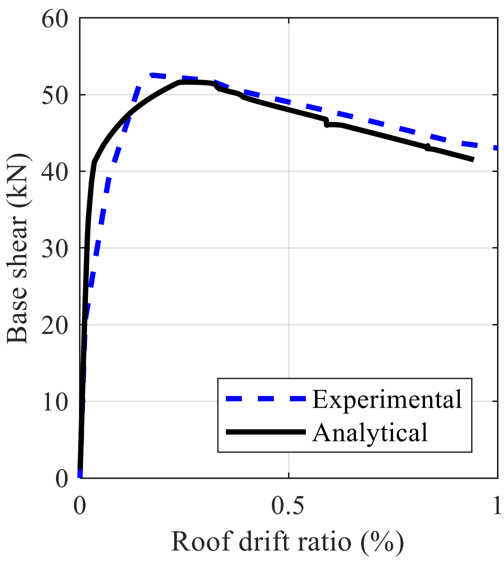


(a) Schematic of an infilled RC frame

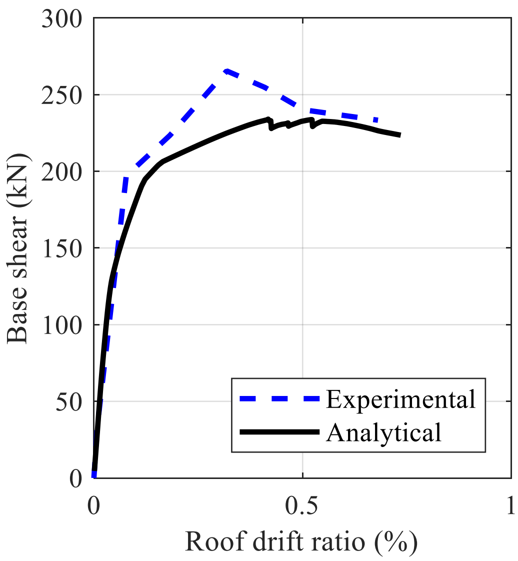


(b) Schematic of macro-model (adapted from Ghosh and Kumar [11]; figure reused with permission from The Masonry Society and 13th North American Masonry Conference)

Fig. 2 – A schematic of the analytical model



(a) Specimen 2S-1B [20]



(b) Three-story-one-bay frame [21]

Fig. 3 – Experimental validation of the macro-model for multi-story frames

Pushover analyses could be carried out for all frames with infill except one (see Fig. 4(b)) till the lateral drift corresponding to which the strength dropped to 80% of the peak lateral strength (e.g., [4]). In such scenarios, the last converged drift point or the drift corresponding to 90% of the peak strength (e.g., [5]) may be considered as peak deformation capacity; the latter approach is adopted in the present study. Peak deformation capacity of the three-storied (nine-storied) frames with masonry infills was between 0.7% – 1%

(0.9% – 1.1%) of the total building height. Convergence issues were encountered for bare frames at drifts between 2.5% – 3.5% of the total building height.

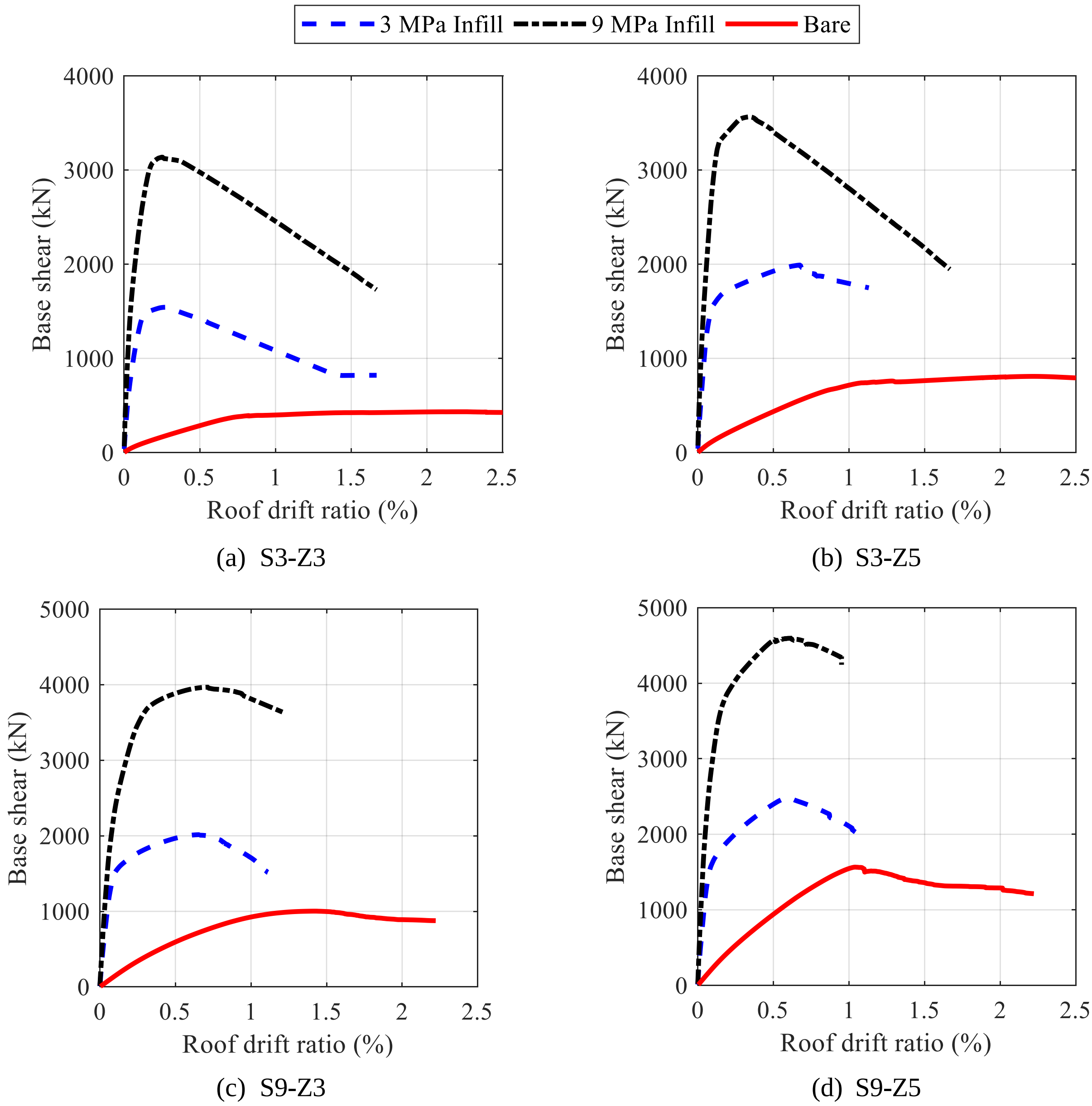


Fig. 4 – Pushover analysis results

Shear failure in the columns of the ground story was observed after the peak lateral force was achieved in the frames. The drifts corresponding to the shear failure decreased with the introduction of masonry struts and increase in the masonry prism strength. As an example, shear failure in an interior column was observed at a roof drift of 2.53%, 1.32% and 0.76% for S3-Z3-Bare, S3-Z3-3MPa and S3-Z3-9MPa, respectively, while the drifts corresponding to the peak lateral force in the frames were 2.21%, 0.28% and 0.25%, respectively. Similar observations were made for other frames.

Lateral forces transmitted through columns increased with an increase in masonry prism strength. Corresponding to the peak lateral force in the three-story frames, the magnitude of lateral force resisted by the four columns together in the ground story was 433 kN, 573 kN and 1,002 kN for frames S3-Z3-Bare, S3-Z3-3MPa and S3-Z3-9MPa, respectively. Total design shear force of the four columns in the ground story was 318 kN. Therefore, an increase in strength of masonry may make the columns vulnerable to shear failure. Similar observations were made for other frames studied.

Select results of pushover analyses are summarized in Table 3. Parameters V_{max} and $RDR_{20\%}$ ($RDR_{10\%}$) are maximum base shear and roof drift ratio corresponding to 20% (10%) loss in the maximum base shear. Elastic fundamental period (T_1) for each frame is also presented in Table 3. Parameter V_{des} in Table 3 is design shear force for all four columns in the ground story.

Table 3 – Pushover and incremental dynamic analyses results

Frame ID	T_1 (s)	V_{max} (kN)	V_{des} (kN)	V_{max}/ V_{des}	$RDR_{20\%}$ ($RDR_{10\%}$) (%)	S_{MT} (g)	Median CIDR (%)	S_{CT} (g)	CMR (S_{CT}/ S_{MT})
S3-Z3-Bare	0.331	433	373	1.16	–[1]	–	–	–	–
S3-Z3-3MPa	0.094	1,543	373	4.14	0.74 (0.55)	0.39	0.35	2.04	5.29
S3-Z3-9MPa	0.072	3,137	373	8.41	0.94 (0.65)	0.33	0.87	4.18	12.67
S3-Z5-Bare	0.292	809	839	0.96	–[1]	–	–	–	–
S3-Z5-3MPa	0.091	1,991	839	2.37	–[2] (0.93)	0.85	0.47	2.46	2.89
S3-Z5-9MPa	0.066	3,566	839	4.25	0.97 (0.67)	0.72	0.58	5.48	7.65
S9-Z3-Bare	0.92	1,003	1,118	0.90	–[1]	–	–	–	–
S9-Z3-3MPa	0.328	2,015	1,118	1.80	1.06 (0.91)	0.40	0.29	1.35	3.38
S9-Z3-9MPa	0.262	3,967	1,118	3.55	–[2] (1.22)	0.40	0.21	2.23	5.58
S9-Z5-Bare	0.748	1,566	2,516	0.62	–[1]	–	–	–	–
S9-Z5-3MPa	0.302	2,482	2,516	0.99	1.07 (0.87)	0.90	0.32	1.53	1.70
S9-Z5-9MPa	0.219	4,595	2,516	1.83	–[2] (0.95)	0.90	0.19	2.92	3.24

[1] It was not possible to calculate $RDR_{20\%}$ for bare frames as the lateral force corresponding to was greater than 80% of the maximum lateral force at the maximum drifts achieved during the analysis.
[2] Pushover analysis stopped before the lateral force dropped to 80% of peak lateral force.

6. Ground motions for incremental dynamic analyses

Twenty-two pairs of ground motion records listed in FEMA P695 [4] are used to perform incremental dynamic analyses (IDA). These ground motions were recorded during earthquakes with magnitudes between 6.5 and 7.6, and at sites located between 8 and 100 km from epicenter. The ground motions were amplitude-scaled so that the peak ground velocity (PGV) for each of the 44 records was identical (= 41.5 cm/s). Five percent-damped response spectra of these scaled motions are presented in Fig. 5. Also shown in the figure is the median response spectrum for these 44 records. Fig. 5 also presents the maximum considered earthquake (MCE) spectra for seismic zones III and V according to the Indian earthquake code IS 1893 [13], considering medium soil condition and an importance factor of 1.0.

7. Collapse capacity assessment

Incremental dynamic analyses (e.g., [23]) were performed for infilled frames identified in Table 1 subjected to ground motions of Fig. 5 (approximately 7,500 response-history analyses). Fig. 6(a) presents spectral accelerations plotted against the maximum IDR among all stories for frame S3-Z3-3MPa subjected to each of the 44 ground motion records scaled to different intensity levels. These curves are known as IDA curves. The point on an IDA curve for which the slope is 20% of the initial slope of the curve represents collapse for bare frames (e.g., [24]). However, as noted by Jeon et al. [22], initial stiffness of the infilled frames is considerably high compared to a bare frame and 20% of initial slope may lead to an underestimation in the collapse inter-story drift ratio (CIDR). Therefore, the definition of the CIDR can be based on 10% of the initial slope of the IDA curve. This definition of CIDR is considered in the present study. The median value of CIDR was 0.35% for frame S3-Z3-3MPa. Also shown in Fig. 6(a) are the median, and 16th and 84th percentile IDA curves. Fig. 6(b) present these results for frame S3-Z3-9MPa. Fig. 7 presents the IDA curves for infilled frames designed for seismic zone V. Key parameters from Figs. 6 and 7, namely, S_{CT}, S_{MT} and median CIDR, are presented in Table 3, where S_{CT} is defined as the spectral acceleration corresponding to the elastic fundamental period at which collapse is achieved for at least 50% of the ground motion records and S_{MT} is defined as the MCE spectral acceleration corresponding to the elastic fundamental period.

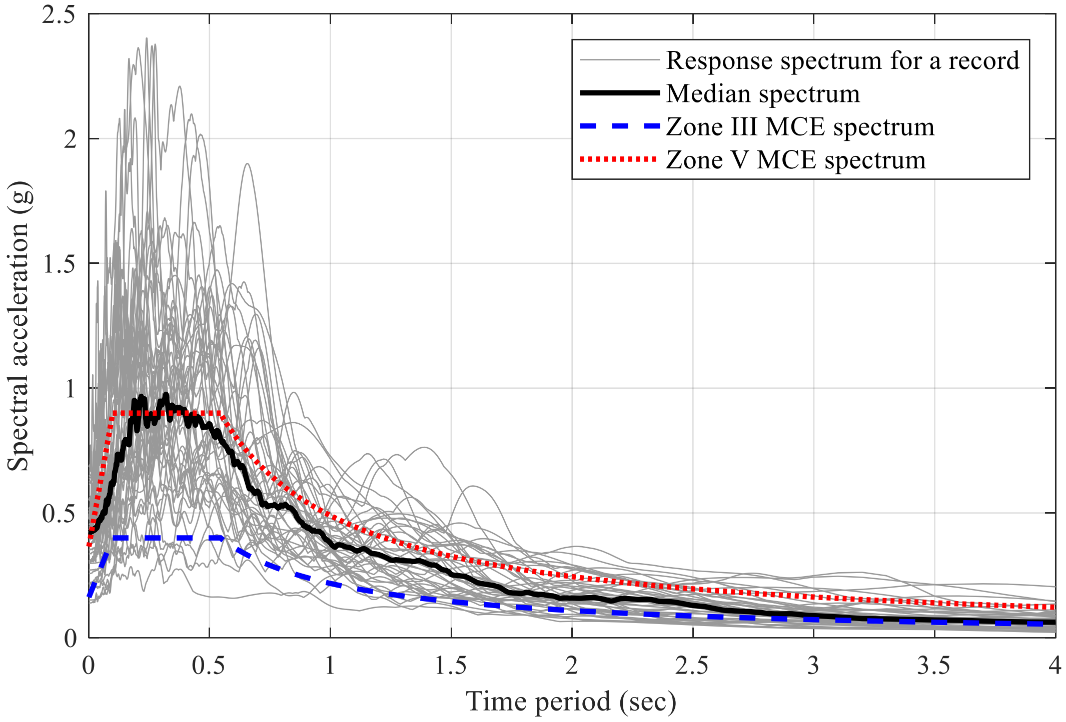


Fig. 5 – Response spectra of the scaled motions considered in the present study

The procedure to assess collapse performance presented in FEMA P695 [4] is considered in the present study, wherein the collapse margin ratio (CMR) is defined as the ratio of S_{CT} to S_{MT}. The ratio can be seen as a factor by which MCE intensity for a seismic zone must be increased to achieve the 50% probability of collapse for a structural system. These ratios for the infilled frames identified in Table 1 are presented in Table 3. Collapse margin ratio for infilled frames increased with an increase in the masonry prism strength. As an example, the CMR for frame S3-Z5-3MPa was 2.89 which increased to 7.65 for the frame S3-Z5-9MPa. Collapse margin ratio for the infilled frames in seismic zone III is greater than that for frames in seismic zone V. For the nine-storied frames and seismic zone III (V), the CMR is 3.38 (1.70) and 5.58 (3.24) for infilled frames with masonry prism strength of 3 MPa and 9 MPa, respectively. Collapse margin ratio decreases significantly with an increase in the number of stories. As an example, the CMR for the frame S3-Z5-3MPa is 2.89 which reduces to 1.70 for the frame S9-Z5-3MPa.

Expectedly, collapse intensity for frames in seismic zone III is smaller than that for frames in seismic zone V. For example, for the nine-storied frames and seismic zone III (V), the collapse intensities are 1.35g (1.53g) and 2.23g (2.92g) for infilled frame with 3 MPa and 9 MPa infill, respectively.

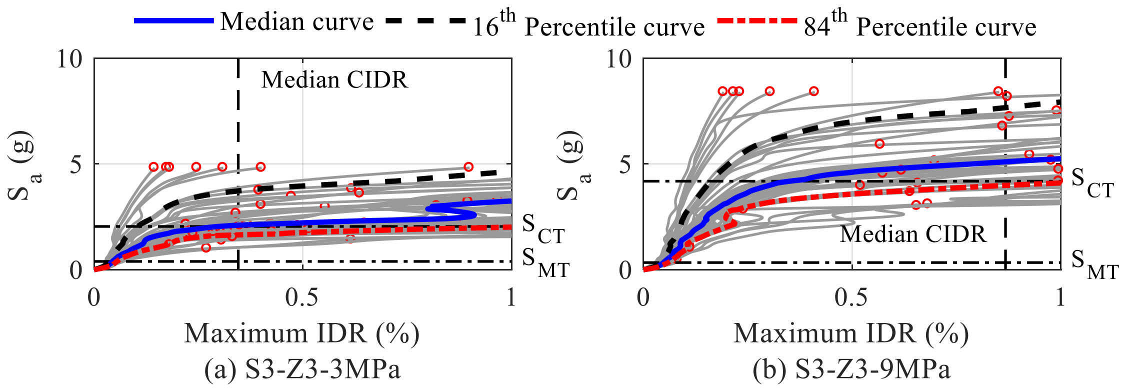


Fig. 6 – Incremental dynamic analysis curves for three-storied frames in seismic zone III

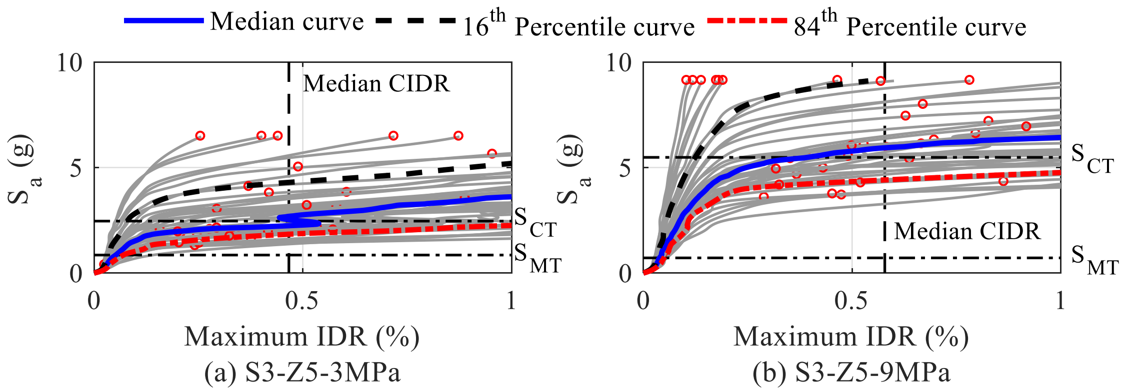


Fig. 7 – Incremental dynamic analysis curves for three-storied frames in seismic zone V

8. Summary and conclusions

This paper presents a study on the bare and masonry-infilled RC frames designed in accordance with the Indian earthquake code. Following parameters are varied for design: number of stories (three and nine), seismic zone (III and V), and masonry prism strength for infilled frames (3 MPa and 9 MPa). A total of 12 frames (including bare frames) are considered. These frames have three bays, each bay 5 m long. Story height is 3 m. A recently developed macro model was used to perform nonlinear static and incremental dynamic analyses. The model was validated against two experimental studies conducted on single-bay-multi-story infilled RC frames.

Presence of infill significantly increases the strength and initial stiffness of the bare frame. The increase in lateral strength is associated with a substantial increase in the shear forces in the ground story columns, which means that not considering the strength of infill at the design stage may be detrimental to the seismic performance of the infilled RC frames. This observation is significant because design codes across the world do not consider the strength of the infill in the design of RC frames.

Increase in masonry prism strength from 3 MPa to 9 MPa increased the collapse margin ratio of infilled frames by 60 – 170%. The collapse margin ratios were considerably smaller for a higher seismic zone or a greater number of stories.

9. Acknowledgements

Authors gratefully acknowledge the financial support provided by the Science and Engineering Research Board (File No. YSS/2015/1514) of the Department of Science and Technology, Government of India.

10. Copyrights

11. References

[1] Kumar M, Rai DC, Jain SK (2015): Ductility reduction factors for masonry-infilled reinforced concrete frames. *Earthquake Spectra*, **31** (1), 339-365.

[2] Mehrabi AB, Benson Shing P, Schuller MP, Noland JL (1996): Experimental evaluation of masonry-infilled RC frames. *Journal of Structural Engineering*, **122** (3), 228-237.

[3] Kaushik HB, Rai DC, Jain SK (2006): Code Approaches to Seismic Design of Masonry-Infilled Reinforced Concrete Frames: A State-of-the-Art Review. *Earthquake Spectra*, **22** (4), 961-983.

[4] Federal Emergency Management Agency (FEMA) (2009): Quantification of building seismic performance factors. *Technical Report FEMA P695*, Federal Emergency Management Agency, Washington, DC.

[5] Sattar S, Liel AB (2016): Seismic performance of nonductile reinforced concrete frames with masonry infill walls—II: collapse assessment. *Earthquake Spectra*, **32** (2), 819-842.

[6] Sattar S, Liel AB (2010): Seismic performance of reinforced concrete frame structures with and without masonry infill walls. *9th US National and 10th Canadian Conference on Earthquake Engineering*, Toronto, Canada.

[7] Pisode M, Surana M, Haldar P, Singh Y (2017): Comparative Assessment of Seismic Fragility of RC Frame Buildings designed for Older and Revised Indian Standards. *ISET Journal of Earthquake Technology*, **54** (1), 17-29.

[8] Burton HV, Sharma M (2019): Quantifying the Reduction in Collapse Safety of Main Shock–Damaged Reinforced Concrete Frames with Infills. *Earthquake Spectra*, **33** (1), 25-44.

[9] Suthasit M, Warnitchai P (2018): Seismic Performance of US-Code Conforming RC Moment Frames Constructed with Regularly Distributed Masonry Infills. *Journal of Earthquake Engineering*, 1-29

[10] Ghosh R, Kumar M (2020): Macro models for in-plane lateral response of reinforced concrete frames with masonry infills. (*under review*)

[11] Ghosh R, Kumar M (2019): Evaluation of Macro Models for Masonry-Infilled Reinforced Concrete Frames. *13th North American Masonry Conference*, Salt Lake City, Utah.

[12] BIS (1987): Code of practice for design loads (other than earthquake for buildings and structures: Part 2 Imposed loads. *IS 875*, New Delhi.

[13] BIS (2016): Indian Standard Criteria for Earthquake Resistant Design of Structures: Part 1, General Provisions and Buildings. *IS 1893*, New Delhi.

[14] Jain A (2020): Collapse performance assessment of RC frame buildings with masonry infills. M.Tech. Thesis, Indian Institute of Technology Gandhinagar, India

[15] Pacific Earthquake Engineering Research Center (PEER) (2014): Open System for Earthquake Engineering Simulation (Version 2.4.3). *Computer program*, Berkeley, California, USA.

[16] Mander JB, Priestley MJ, Park R (1988): Theoretical stress-strain model for confined concrete. *Journal of Structural Engineering*, **114** (8), 1804-1826.

[17] Elwood KJ, Moehle JP (2005): Drift capacity of reinforced concrete columns with light transverse reinforcement. *Earthquake Spectra*, **21** (1), 71-89.

[18] Sezen H, Moehle JP (2004): Shear strength model for lightly reinforced concrete columns. *Journal of Structural Engineering*, **130** (11), 1692-1703.

[19] Mainstone RJ (1971): On the stiffness and strengths of infilled frames, *Proceedings of Institution of Civil Engineers*, Supplement **IV**, 57–90.

[20] Suzuki T, Choi H, Sanada Y, Nakano Y, Matsukawa K, Paul D, Gülkan P, Binici B (2017): Experimental evaluation of the in-plane behaviour of masonry wall infilled RC frames. *Bulletin of Earthquake Engineering*, **15** (10), 4245–4267.

[21] Koutas L, Bousias SN, Triantafillou TC (2015): Seismic strengthening of masonry-infilled RC frames with TRM: Experimental study. *Journal of Composites for Construction*, **19** (2), 04014048.

[22] Jeon JS, Park JH, DesRoches R (2015): Seismic fragility of lightly reinforced concrete frames with masonry infills. *Earthquake Engineering & Structural Dynamics*, **44** (11), 1783-1803.

[23] Vamvatsikos D, Cornell CA (2002): Incremental dynamic analysis. *Earthquake Engineering & Structural Dynamics*, **31** (3), 491-514.

[24] Vamvatsikos D, Cornell CA (2002): The incremental dynamic analysis and its application to performance-based earthquake engineering. *12th European Conference on Earthquake Engineering*, London, UK.

