Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 583

SEISMIC PERFORMANCE OF REINFORCED CONCRETE FRAME STRUCTURES WITH AND WITHOUT MASONRY INFILL WALLS

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ABSTRACT

Nonductile reinforced concrete frames with masonry infill walls are a popular form of construction in seismic regions worldwide. This study assesses the seismic performance of these buildings, utilizing dynamic analysis of nonlinear simulation models to obtain probabilistic predictions of the risk of structural collapse. The evaluation is based on structures with design and detailing characteristics representative of pre-1975 California construction.

This research quantifies the effect of the presence and configuration of masonry infill walls on seismic collapse risk. Seismic performance assessments indicate that, of the configurations considered (bare, partially-infilled and fully-infilled frames), the fully-infilled frame has the lowest collapse risk and the bare frame is found to be the most vulnerable to earthquake-induced collapse. Depending on the infill configuration, the median collapse capacity varies by a factor of 1.3 to 2.5. The results for fully-infilled frames are likely upper bounds for collapse capacity, since they do not account for column shear failure, which may be significant in some cases. The presence of masonry infill also significantly changes the collapse mechanism of the frame structure, leading to a first-story mechanism in most cases. Results are similar for structures of varying heights (4 and 8 stories).

Introduction

Reinforced concrete (RC) frame buildings with masonry infill walls have been widely constructed for commercial, industrial and multi-family residential uses in seismic-prone regions worldwide. Masonry infill typically consists of brick, clay tile or concrete block walls, constructed between columns and beams of a RC frame. These panels are generally not considered in the design process and treated as architectural (non-structural) components. Nevertheless, the presence of masonry walls has a significant impact on the seismic response of an RC frame building, increasing structural strength and stiffness (relative to a bare RC frame), but, at the same time, introducing brittle failure mechanisms associated with the wall failure and wall-frame interaction.

The present study assesses the seismic performance of RC frame structures with masonry infill walls, utilizing dynamic analysis of nonlinear simulation models to obtain probabilistic predictions of the risk of structural collapse. In particular, the research examines how the presence and configuration of masonry infill walls affects seismic collapse risk and seeks to identify design characteristics of vulnerable infilled RC frames. Previous research has shown that some older (nonductile) RC frame structures have substantially higher risks of earthquake-induced collapse than other structures, potentially endangering the safety of building occupants

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(Liel et al. 2010). However, this research did not examine the buildings with masonry infill panels, and the walls' effect on seismic collapse risk is uncertain.

Seismic performance is predicted from nonlinear simulation models of nonductile RC frame buildings with masonry infill walls. Design and detailing characteristics of case study buildings are based on typical pre-1975 California 4 and 8-story frame buildings with different infill configurations. Models for masonry walls and RC beams, columns and joints have material and geometric nonlinear properties for simulation of sidesway collapse. Collapse is predicted through incremental dynamic analysis of nonlinear simulation models. This analysis provides metrics of seismic safety for this type of building.

Background on RC Frames with Masonry Infill

This study builds on a number of experimental and analytical efforts to evaluate the effect of masonry infill panels on the seismic behavior of frame structures. Polyakov (1960) conducted experimental tests on masonry-infilled frames, first proposing that the infill system works as a braced frame, with the wall forming compression "struts". Following this approach, Stafford-Smith (1962) and Mainstone (1971), among others, proposed methods for calculating the effective width of the diagonal strut, supported by test results from mortar panels and infilled frames, respectively. Other experiments examined the performance of infilled frame structures more broadly. Klingner and Bertero (1978) tested a one-third scale 3.5 story representation of an 11-story 1970s-era RC apartment building. Their study concluded that reinforced infill panels reduce the risk of incremental collapse, compared to a bare RC frame. Mehrabi et al. (1996) tested twelve ½-scale single-story single-bay frame specimens and observed that the frames with infill showed better seismic performance than the bare frames.

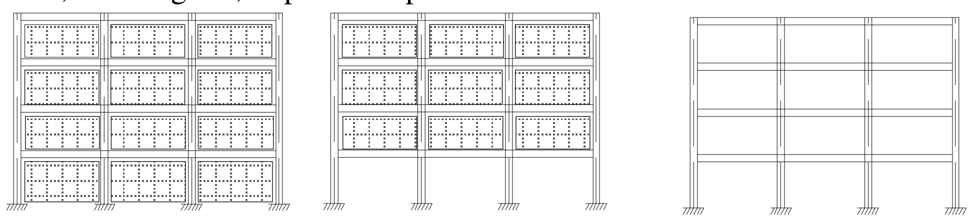
Analytical methods to model masonry infill panels have advanced alongside experimental research. Based on infill tests by Polyakov (1960) and others, Holmes (1961) proposed a linear equivalent strut model for computing maximum strength and stiffness of masonry walls. Stafford-Smith and Carter (1969) developed analytical techniques to calculate the effective width of the strut, and cracking and crushing loads, as a function of the contact length between frame and wall elements. Flanagan and Bennett (1999) used a piecewise-linear equivalent strut to model infill and proposed an analytical procedure to calculate the strength of the infill, based on experimental results of 21 steel frames with clay tile infill walls. Other researchers have used finite-element models to represent complex aspects of wall behavior. Dhanaskar and Page (1986) modeled an infilled frame using nonlinear finite brick elements, comparing the results with several half-scale experiments. Mehrabi and Shing (1997) used a smeared-crack finite element model to represent masonry units and RC frames, developing a constitutive model for mortar joints. Stavridis and Shing (2009) have developed a complex nonlinear finite element model for RC frames with masonry infill, combining the smeared and discrete crack approaches to capture different failure modes observed in experiments.

More recent research has combined analytical and experimental methods to evaluate the seismic performance of RC frames with masonry infill more generally. Dolsek and Fajfar (2008) used concentrated plasticity beam-column model elements with equivalent strut wall elements to evaluate the seismic performance of masonry-infilled RC frames, looking at "damage limitation", "significant damage" and "near collapse" limit states. Dymiotis et al. (2001) assessed the seismic vulnerability of a 10-story infilled RC frame at "serviceability" and "ultimate" limit states. Madan and Hashmi (2008) evaluated the performance of 7 and 14-story RC frames with masonry infill subjected to near-fault ground motions.

The assessment of RC frames with masonry infill panels here simulates structural collapse, evaluating life safety with nonlinear models and limit-state checks. Performance-based earthquake engineering techniques account for uncertainties in ground motions and modeling.

Case Study Buildings

This study is based on structures with design and detailing characteristics typical of pre-1975 California construction. The paper focuses on nonductile RC frame structures of two heights, 4 and 8 stories. Each building is evaluated as a bare frame and with two different infill configurations, as shown for the 4-story structure in Fig.1. Infill is commonly omitted at the ground floor, as in Fig. 1b, to provide open windows for retail and commercial services.



(a) Fully-Infilled (b) Partially-Infilled (c) Bare Frame

Figure 1. Masonry infill configurations for case study RC frames.

Design and detailing of the RC frame structures and constituent elements have been described in Liel et al. (2010). Columns are spaced at 25 ft. and story heights are 15 ft. in the first-story and 13 ft. in upper stories. These buildings are perimeter frame systems, with flat-slab interior gravity systems. The RC frames are designed to resist seismic loads associated with the highest seismic zone in the 1967 Uniform Building Code. Reinforcement detailing is typical of that provided before seismic detailing rules were established, and includes widely spaced transverse reinforcement, poor anchorage of stirrups and longitudinal reinforcement, and no joint shear reinforcement. The buildings are assumed to be located at a high seismic site with maximum considered earthquake (MCE) of S_{MS} = 1.5g and S_{M1} = 0.9g.

Masonry material properties for the single-wythe walls are based on data from Manzouri (1995) for bricks reclaimed from an old building with mortar type "O". The nominal width of the masonry is 4.125 in. and the thickness of mortar joints is 0.375 in. These properties represent a weak type of masonry infill, like that used in past construction. This masonry has compressive strength (f'_m) of 2000 psi and shear strength (f'_v) varying from 120 to 270 psi, depending on the normal stress. The Young's modulus of masonry (E_m) is 580 psi.

Nonlinear Models for RC Frame and Masonry Infill

Nonlinear analysis models for the RC frame structures consist of the two-dimensional three-bay frame, as shown in Fig. 2. Models are implemented in *OpenSees*. The simulation model captures material nonlinearities in beams, columns, beam-column joints, and masonry walls. A leaning column accounts for the additional seismic mass on the gravity system (P-Δ effects), but not the contribution of the gravity system to the lateral resistance of the frame. Deterioration in the beams, columns, and joints is modeled with concentrated springs idealized by the tri-linear backbone and associated hysteretic rules developed by Ibarra et al. (2005). An important attribute of this backbone is the post-peak negative stiffness, which enables modeling of strain softening behavior associated with concrete crushing, rebar buckling and fracture as collapse occurs. Properties of the inelastic springs representing beam and column elements are calibrated to mean values from experimental tests of 255 columns (Haselton et al. 2008). These structural models, together with nonlinear geometric transformations and robust convergence algorithms, are capable of representing structural response into the collapse limit state. The primarily limitation of the frame model is that column shear failure is not explicitly represented, due to difficulties in accurately representing this limit state and the subsequent loss of gravity-load bearing capacity. RC column models do capture degradation in shear strength associated with flexure-shear failure (i.e. yielding followed by shear failure).

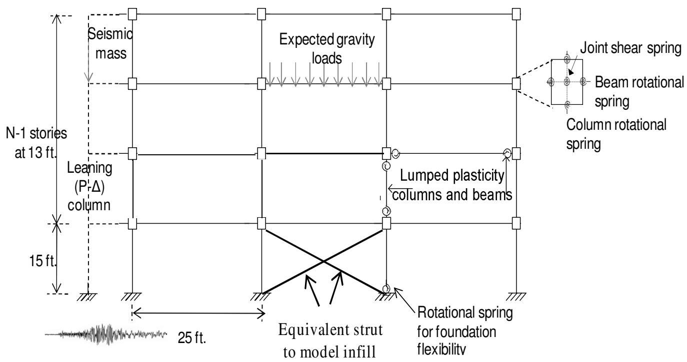


Figure 2. Nonlinear analysis model for RC frame building with masonry infill walls.

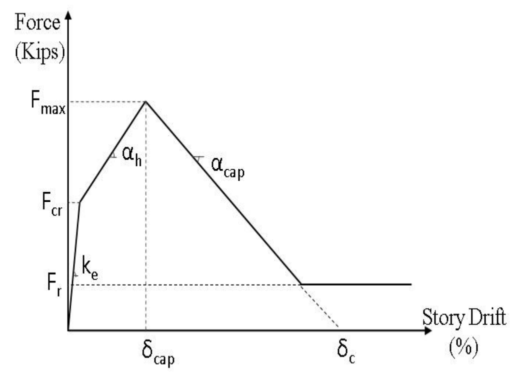


Figure 3. Modeled force displacement behavior for infill strut material.

Each infill panel is simulated with a pair of compression struts, as illustrated in Fig. 2. Inelastic struts are used to represent infill walls because they have sufficient accuracy to capture key characteristics of force-displacement response, but reduce computational effort in comparison to the finite element models. Each strut is assigned a force displacement relationship (shown in Fig. 3) representing initial stiffness, peak strength, and post-peak behavior of the masonry necessary to predict wall failure.

The properties of the modeled equivalent struts are as follows. The equivalent strut width, w, is computed from Stafford-Smith and Carter (1969) based on a set of curves relating w to a non-dimensional parameter, λh, expressing the relative stiffness of the frame to the infill, where

$$\lambda h = h\left(\frac{E_m\, t \sin(2\theta)}{4\, E\, I\, h'}\right)^{0.25} \tag{1}$$

and E_m, t, and h' are Young's modulus, thickness, and height of the infill, respectively. Properties of the columns are E (Young's modulus) and I (moment of inertia). θ is the angle between diagonal of the infill and the horizontal. The initial stiffness of the masonry infill panel, k_e, is taken as twice the stiffness obtained from the equivalent strut width and the properties of the masonry, i.e. $k_e = 2(E_m wt / L)(\cos\theta)^2$, where L is the length of the diagonal strut.

After cracks form in the infill panel, the stiffness of the panel reduces to $\alpha_h k_e$, but the force in the panel continues to increase until failure occurs due to shear, sliding, or crushing. This study uses the equation proposed by Zarnic and Gostic (1997) and later modified by Dolsek and Fajfar (2008) to determine the maximum strength of the infill:

$$F_{\max} = 0.818 \frac{L_{in}\, t\, f_{tp}}{C_I}(1+\sqrt{C_I^{\,2}+1}), \qquad C_I = 1.925\frac{L_{in}}{h'} \tag{2}$$

where L_{in} is the length of the infill, f_{tp} is the cracking stress of the masonry and t and h' are defined as before. This equation has been validated by a set of experiments conducted on single-bay single-story and two-bay two-story infilled frames by Zarnic and Gostic (1997). For the panels and masonry properties considered in the frames modeled here, the maximum strength of the infill of each wall is approximately 141 kips. The ratio of cracking force to maximum strength (F_{cr}/F_{max}) is taken as 0.55, following experimental data from Manzouri (1995) and recommendations from Dolsek and Fajfar (2008). Struts have negligible tensile strength.

The deformation capacity of the infill panel is based on recommendations from past researchers and observations from experimental tests. Manzouri (1995)'s experimental results show that the displacement at the maximum load (i.e. δ_{cap} in Fig. 3) occurs at approximately 0.25% drift. Likewise, a set of experiments conducted by Shing et al. (2009) found that the maximum load in the frame specimen occurs at 0.25% drift. Based on this evidence, δ_{cap} is taken here as 0.25% drift. Post-peak strength degradation (represented by α_c in Fig. 3) is based on Dolsek and Fajfar's (2008) estimate that the displacement at zero wall strength (δ_c) is approximately five times the displacement at maximum force (δ_{cap}). The infill characteristic parameters proposed by Dolsek and Fajfar (2008) are supported by a set of experiments reported in Carvalho and Coelho, Eds. (2001). The residual strength of the wall (F_r) is assumed to be 20% of the maximum strength. This is a conservative value, based on experimental data that shows that wall strength after failure varies from approximately 30% to 60% of the maximum strength (FEMA 1998). Cyclic deterioration of the infill panels is not considered due to lack of data; since the panels fail abruptly under relatively small deformations, cyclic degradation is not expected to dominate the response so this does not present a significant limitation.

Static Pushover Analysis Results

Results from static pushover analysis for the case study buildings are shown in Fig. 4 and summarized in Table 1. Lateral loads were applied according to the equivalent lateral force distribution specified in ASCE 7-05.

The presence of the infill wall both strengthens and stiffens the system, as illustrated in Fig. 4. For the 4-story building, the fully-infilled frame has approximately 15 times larger stiffness and 1.5 times greater peak strength than the bare frame. In Fig. 4a, the first drop in strength for the fully-infilled frame is due to the brittle failure of masonry materials initiating in the first-story infill walls. Lateral loads are subsequently redistributed and the pushover curve reaches its peak strength again as the walls in upper stories and frame elements are subjected to higher loads. This behavior after first-story wall failure is due to wall-frame interaction and depends on the relative strength of the infill and framing. The response of the partially-infilled frame is between the fully-infilled and bare frame case. Unlike the fully-infilled RC frame, the partially-infilled response does not exhibit brittle wall failure, due to the absence of walls in the first story. Comparing Fig. 4a and 4b, the 8-story frame buildings are consistently stronger, because they are designed for higher seismic loads (higher effective mass). In the 8-story structures, the sudden post-peak drop in base shear occurs at a smaller maximum deformation capacity, due to the increased importance of P-Δ effects in taller, more flexible buildings.

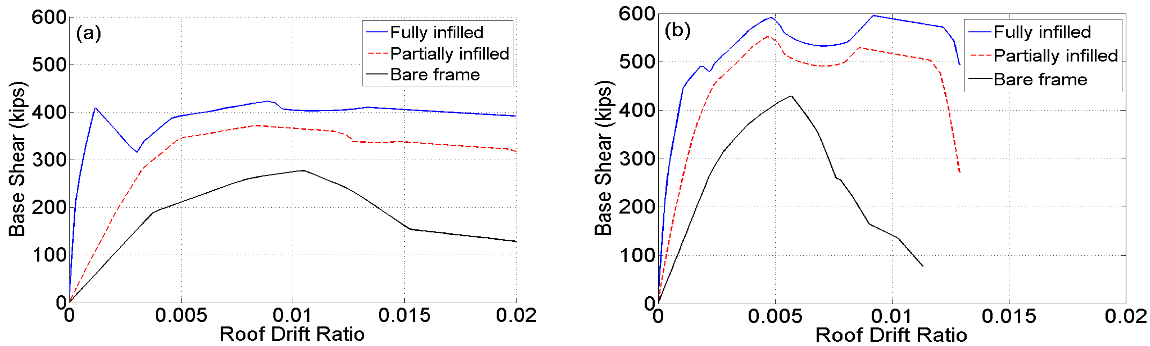


Figure 4. Pushover analysis results for (a) 4-story and (b) 8-story RC frame buildings.

Seismic Performance Assessment

The procedure for assessing seismic performance applies the methodology for performance-based earthquake engineering developed by the Pacific Earthquake Engineering Research Center, which provides a probabilistic framework for relating ground motion intensity to structural response and building performance through nonlinear time-history simulation. Incremental Dynamic Analysis (IDA) is used to assess global sidesway collapse (Vamvatsikos and Cornell 2002). In IDA, the nonlinear structural model is subjected to a recorded ground motion, and dynamically analyzed to predict the structure's response. The time-history analysis is repeated, each time increasing the scale factor on the input ground motion, until that record causes structural collapse, as identified by interstory drifts that increase without bounds (i.e. dynamic instability). This process is repeated for a large set of ground motion records, in order to quantify record-to-record variation in nonlinear structural response. This study uses 44 recorded ground motions (22 pairs) selected to represent large earthquakes with moderate fault-rupture distances (i.e., not near-fault conditions) (FEMA 2009). The outcome of IDA is a fragility function, a cumulative probability distribution that defines the probability of structural (simulated) collapse as a function of the ground motion intensity (given by the spectral acceleration at the first mode period of the building [$Sa(T_1)$]).

IDA results are illustrated in Fig. 5 for the 4-story partially-infilled RC structure, showing the relationship between ground motion intensity and peak inter-story drift ratio for the suite of ground motions. Results are shown for only one horizontal component from each pair of ground motions; the worst-case component in each case is taken to represent three-dimensional effects.

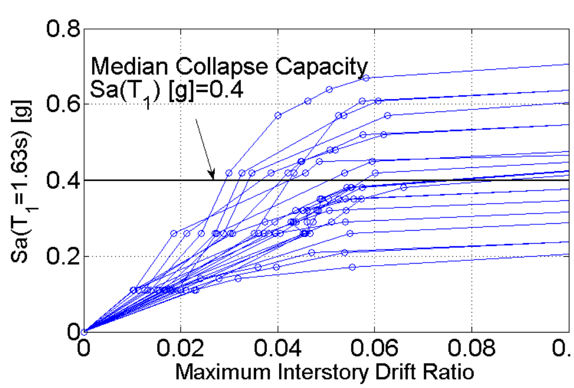


Figure 5. IDA results for 4-story partially-infilled RC frame.

Seismic Performance of Masonry-Infilled RC Frames

The results of the seismic performance assessment are summarized in Table 1 for all case study RC frame buildings. Since the median collapse capacity is a function of the building period, results are normalized by the site-specific *MCE* (at the first mode period) for comparison between different structures. Since ground motions were selected without consideration of spectral shape, the median collapse capacity is adjusted to reflect the expected spectral shape of rare California ground motions following the recommendations of Haselton et al. (2009). For the buildings considered, the final assessment of collapse resistance shows that the ratio of the median collapse capacity to the *MCE* ranges from 0.6 to 1.7. These results indicate that these buildings are substantially less safe than modern seismically-detailed RC frames, which have collapse capacities approximately two times the *MCE*, i.e. a significant safety factor (Liel et al. 2010; FEMA 2009). The collapse risk of each of the RC frames is also represented by a collapse fragility function, as shown in Fig. 6. The fragility functions account for uncertainties due to record-to-record variation in ground motions ($\sigma_{ln,RTR}$) and structural modeling, where $\sigma_{ln,modeling}$ = 0.5 based on previous research by Liel et al. (2009).

Table 1 and Fig. 6 show that the fully-infilled frames have the highest collapse safety of the three infill configurations considered. Compared to the bare frame, the fully-infilled frames have median collapse capacities approximately two times larger, indicating a lower risk of earthquake-induced collapse (smaller probability of collapse). The fully-infilled frames exhibit better seismic performance due to the added strength of the walls in the system. However, it is important to note that the fragility curves proposed in Fig. 6 show the upper bound collapse resistance for infilled frames because the models do not consider the shear failure of the columns, which may be significant in some infilled structures (but not all, see e.g. Dolsek and Fajfar 2008). Partially-infilled frames benefit (see Fig. 4) from increased strength in upper stories due to the presence of the infill walls, but suffer from strength and stiffness discontinuities between the first story and those above, which decreases collapse performance. Table 1 also shows that record-to-record variability ($\sigma_{ln,RTR}$) in seismic collapse performance is much larger for the fully-infilled RC frames than the other buildings. This variation is due to scaling at the short first-mode periods of the fully-infilled buildings in IDA, which induces large variability in spectral values at longer periods. The tendency of the fully-infilled structure to experience a variety of different failure modes (discussed below) could also increase the variability. Trends are similar in 4 and 8-story buildings, though the 8-story buildings tend to have slightly smaller median (normalized) collapse capacities. The worse performance of the 8-story buildings is likely due to increased dominance of P-Δ effects.

The presence of masonry walls has a significant effect on the collapse mechanism observed. To illustrate, Fig. 7 presents the dominant failure modes of bare frame, fully-infilled and partially-infilled frames for the 4-story building. The failure modes shown are the collapse mode experienced most frequently during the 44 ground motion records. The fully-infilled frame (Fig. 7b) fails in a soft first-story mechanism in 82% of the records, though some damage is experienced in second-story columns and walls. When subjected to other ground motion records, the fully-infilled frame experienced different failure modes, such as multi-story (distributed damage) mechanism. The partially-infilled frame (Fig. 7c) consistently fails in a soft-story mechanism (98% of ground motions), due to the much lower strength and stiffness of the first floor because of the configuration of infill walls. Some damage to columns and walls is also observed in the second story. The collapse mechanism of the bare frame (Fig. 7a), typically involves at least two stories, including yielding in columns and joint damage. A two-story mechanism is observed in 75% of the records. Similar patterns of collapse mechanisms were observed for the 8-story buildings.

Table 1. Pushover and IDA results for RC frame buildings with masonry infill walls.

Building	Pushover Results		IDA Results				
	T_1(sec)	Max Base Shear (kips)	Median collapse Capacity($Sa(T_1)$ [g])	$\sigma_{ln,RTR}$	Median Collapse Capacity/MCE	IDR[1]	RDR[2]
4-Story RC Frame							
Bare Frame	1.96	276.8	0.32	0.39	0.70	0.037	0.017
Infill	0.37	423.0	2.49	0.62	1.66	0.065	0.022
Partial Infill	1.62	371.0	0.54	0.34	0.96	0.056	0.019
8-Story RC Frame							
Bare Frame	2.36	429.5	0.23	0.36	0.60	0.034	0.009
Infill	0.78	592.0	1.75	0.59	1.52	0.059	0.013
Partial Infill	1.77	552.0	0.56	0.41	1.10	0.055	0.011

[1]Inter story drift ratio at collapse
[2]Roof drift ratio at collapse

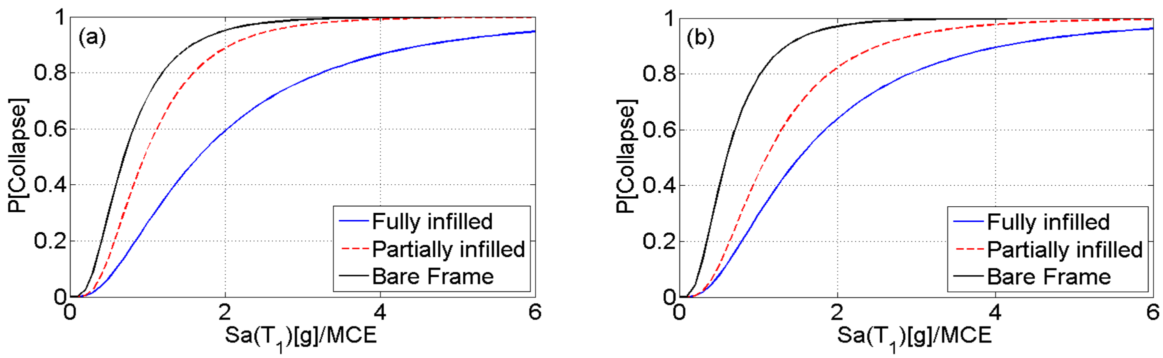
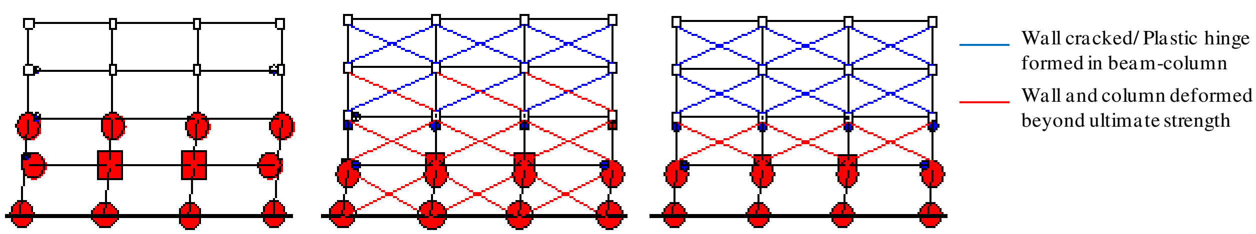


Figure 6. Collapse fragility functions for (a) 4-story and (b) 8-story RC frame structures with different infill configurations.



(a) Bare Frame (b) Fully-Infilled (c) Partially-Infilled

Figure 7. Typical failure modes observed for case study RC frames.

This study agrees with the results of Madan and Hashmi (2008), who observed that the infilled frames experienced less damage than either the bare frame or partially-infilled buildings, due to higher stiffness and strength. However, their study did not scale ground motions to levels to induce earthquake collapse. The superior performance of infilled buildings is also consistent with past experimental results (e.g. Klingner and Bertero 1978). The simplified performance assessment presented byDolsek and Fajfar (2008) found that bare RC frames have higher annual probability of exceeding the near-collapse limit state than either the fully-infilled or partially-infilled frame. Our results show that the performance of the partially-infilled frame is between fully-infilled and bare frame, at least for the case study buildings considered. Differences in collapse performance for fully-infilled and partially-infilled frames are more significant here than observed by Dymiotis et al. (2001). Their study also showed smaller likelihood of failure for bare than infilled frames, but they acknowledge that this finding generally does not agree with field observations.

Limitations and Future Work

Consideration of column shear failure will likely decrease the predicted median collapse capacity of the infilled frames. Work is ongoing to incorporate shear and subsequent axial failure models into frame simulations. In addition, due to the uncertainty in wall modeling parameters, sensitivity analyses will be conducted to identify those modeling parameters that have the greatest impact on seismic performance assessment. Preliminary results suggest that maximum strength, the post-capping slope, and residual strength are the most important wall modeling parameters. Further studies will examine the effect of different types of masonry materials on seismic performance, and evaluate the behavior of frames with stronger walls and infill with openings for windows and doors.

Conclusions

This study assesses the seismic performance of masonry-infilled RC frames, including a set of 4 and 8-story buildings with different infill configurations. Infill panels are modeled by two nonlinear strut elements, which only have compressive strength. Nonlinear models of the frame-wall system are subjected to incremental dynamic analysis in order to assess seismic performance.

Results of pushover analysis show an increase in initial stiffness, strength, and energy dissipation of the infilled frame, compared to the bare frame, despite the wall's brittle failure modes. Likewise, dynamic analysis results indicate that fully-infilled frame has the lowest collapse risk and the bare frames are found to be the most vulnerable to earthquake-induced collapse. The better collapse performance of fully-infilled frames is associated with the larger strength and energy dissipation of the system, associated with the added walls. Similar trends are observed for both the 4- and 8-story RC frames.

Acknowledgments

The authors would like to thank Professor Richard Klingner and Shiv Shanker at UT Austin for their advice on modeling masonry infill walls in *OpenSees*.

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