Unified Analysis on Progressive and Seismic Collapses of RC Frame Structure: The Effect of Masonry-infill Walls

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SUMMARY:
The two major reasons caused the collapse of reinforced concrete frames are earthquake loads and accidental loads, which are termed as the structural progressive collapse and the seismic collapse. Most of the current studies deal with one of the two collapse patterns, although both of them should be considered in the procedures of structural analysis and design. Nonlinear static analysis and dynamic analysis are applied to reinforced concrete frames with and without masonry-infill walls to explore the structural collapse-resistant capabilities. The contribution of the infill walls to the overall performance of structural progressive collapse and the seismic collapse is considered. According to the analysis results, the infill walls can reduce the displacement responses and increase resistant capability to progressive collapse, but may increase the demand of member forces. The infill walls may lead to a soft-story damage pattern when the structure subjected to earthquake excitations. The progressive collapse and seismic collapse provide different deformation and force demands on structural columns and beams.

Keywords: progressive collapse, seismic collapse, masonry-infill wall, reinforced concrete frame, analysis and design

1. INSTRUCTION

Progressive collapse is referred to the phenomenon of disproportionate collapse of structures in column lose scenario induced by accidental hazards such as gas explosion at Ronan Point in 1968, terrorist attacks on Murrah Federal Building in 1995 and on World Trade Center in 2001. Seismic collapse of structures can be due to deficient lateral load resistance during earthquake. Outcomes of the two structural collapse types often look quite similar. However, the damage mechanisms of progressive and seismic collapses of a structure are not the same.

Many researches on structural progressive collapse and seismic collapse resistances of frames have been done over the last decade. These researches can be roughly catalogued into four aspects. The first aspect focuses on developing numerical analysis methods to investigate the phenomenon of progressive collapse and seismic collapse (Sun *et al.*, 2003; Kaewkulchai and Williamson, 2004; Marjanishvili, 2004; Lynn and Isobe, 2007; Izzuddin *et al.*, 2008; Asgarian and Rezvani, 2012). The second aspect is the studies on various strengthening techniques of structures to resistant collapse (Hayes *et al.*, 2005; Galal and El-Sawy, 2010). The third aspect deals with the structural global performances, i.e., the responses at story level under seismic collapse or responses of residual structure after column loss in progressive collapse (Ibarra, 2003; Khandelwal *et al.*, 2009; Kim and Lee, 2010; Masoero *et al.*, 2010). The last aspect copes with effects of damage of local members on collapse performances, such as effects of joint, slab, catenary action, and masonry-infill walls (Bao *et al*., 2008; Kim *et al.*, 2009; Yu *et al.*, 2010; He, 2010).

The effects of masonry-infill walls, which abovementioned belongs to the third research aspect, recently arises growing interests among researchers. The infill walls are usually considered as nonstructural elements. In conventional analysis and design, only considering the nonstructural elements as loads, the stiffness and strengthen are usually ignored throughout the processes. However, the infill wall may significantly change the collapse resistant potentials and damage patterns. In addition, most of the practical frames contain infill walls rather than bare frames. Unlike many researches on effects of infill walls in the seismic collapse, up to now, there are only a few of researches had been done on effects of infill wall in the progressive collapse. Sadani *et al.* (2008a,b) studied the response of a reinforced concrete (RC) infilled frame to removal of two adjacent columns. This research showed that the development of vierendeel action is a major mechanism in redistribution of loads and the infill walls could decrease the structural maximum displacement after the column removal. Tsai and Huang (2010, 2011) investigated the effects of infill walls, in which they showed that effects of brick infill on progressive collapse resistance depending on its location and dimensions, and the collapse resistance is only slightly influenced by the infill walls due to their brittle failure. Little research has been devoted to systematically and clearly compare the effects of infill walls on progressive collapse resistance with seismic collapse resistance of frames.

To some extent, one certain layout infill walls may enhance one type of collapse resistance, but may aggravate another type of collapse during loading. Hence, there has been growing interest among researchers in studying relationship between progressive collapse and seismic collapse performance of a structure. Hayes *et al.* (2005) investigated the relationship between seismic details and progressive collapse resistance, which showed that by using seismic detailing techniques, strengthening the perimeter members improves progressive collapse resistance. Powell (2005) reviewed the principles of progressive collapse analysis for Alternate Path method, compares static and dynamic analysis methods, and pointed out the differences between progressive collapse and seismic collapse. Bao *et al.* (2008) found that reinforced concrete frames designed for high seismic risk has high progressive collapse resistance than frames designed in low to moderate seismic zones. It is important to identify the difference between progressive collapse and seismic collapse to help of providing an effective way to enhance resistance of structures with or without infill walls.

This study focuses on unified analysis on progressive and seismic collapse performances of frames. The masonry-infill wall effects are considered in the analysis to give a more rational evaluation of practical structures. The different structural performances and capabilities of these frames in progressive and seismic collapses are compared.

2. ANALYTICAL MODEL

Eight-story RC frames with and without infill walls are adopted in the analysis of this study. The plan layout is shown in Fig. 2.1. The story height is 3.3m. The structures were designed according to the Chinese code for seismic design of buildings (GB50011, 2010). Their seismic fortification intensities are equal to 8 (0.2g), and are located in site type II with the first classification of the design earthquake. A dead load of 6kN/m^2 is applied to the floors, and the live load is 2.5kN/m^2 on the floors. The elevation of the transverse frames is shown in Fig. 2.2 with appointed number of structural members located in the first story. B1, B2, and B3 are first-story beams, C1, C4 are first-story corner columns and C2, C3 are first-story centre columns.

Beam element B21 in ABAQUS is used to model beams and columns and fiber cross section is adopted. Bilinear model is used to simulate steel reinforcement. The design yield strength is 388Mpa. The elastic Young's modulus (E_s) is 2×10^5Mpa and plastic modulus is $0.01E_s$. Constitutive model in accordance to GB50010 (2002) is used to simulate concrete subjected to uniaxial compression and tension. Constitutive model of concrete subjected to uniaxial compression is determined by equation (2.1):

$$\frac{\sigma}{f_c^*} = \alpha_a + \left(3 - 2\alpha_a\right)\left(\frac{\varepsilon}{\varepsilon_c}\right)^2 + \left(\alpha_a - 2\right)\left(\frac{\varepsilon}{\varepsilon_c}\right)^3, \quad x \le 1 \tag{2.1a}$$

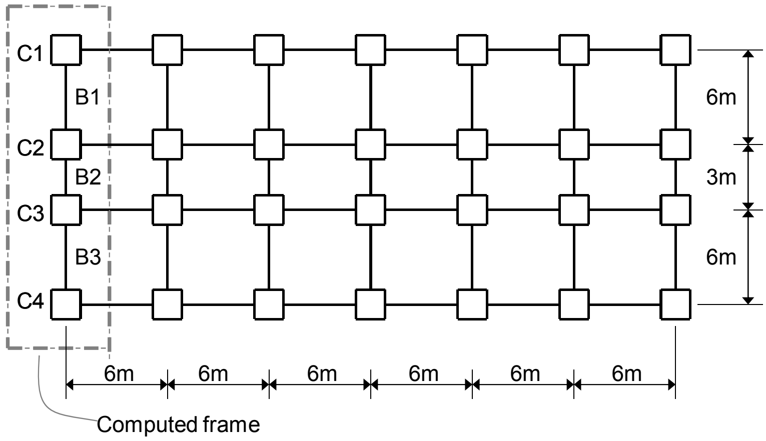
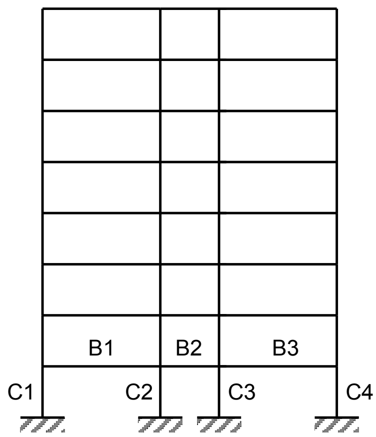
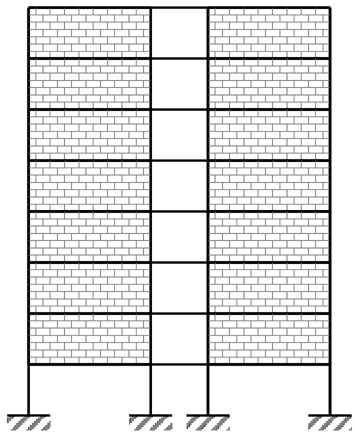


Figure 2.1 Plan layout of the frames



(a) Frame without infill walls



(b) Infilled frame

Figure 2.2 Elevation of frames

$$\frac{\sigma}{f_c^*} = \left(\frac{\varepsilon}{\varepsilon_c}\right) \Bigg/ \left[\alpha_d\left(\frac{\varepsilon}{\varepsilon_c}\text{-}1\right)^2 + \frac{\varepsilon}{\varepsilon_c}\right],\ \ x > 1 \tag{2.1b}$$

where α_a =1.96 and α_d =1.65 are parameters described the ascent stage of constitutive models for concrete subjected to compression, f_c^* =35Mpa is the uniaxial nominal compressive strength, ε_c =0.00172 is the strain corresponding to f_c^*, ε is the concrete strain, σ is the concrete stress, ε_u is the concrete strain corresponding to $0.5 f_c^*$ in descent stage of constitutive model and $\varepsilon_u / \varepsilon_c$ is 2.1 for concrete C35 specified according to GB50010 (2002).

Constitutive model of concrete subjected to uniaxial tension is determined by equation (2.2):

$$\frac{\sigma}{f_t^*} = 1.2\left(\frac{\varepsilon}{\varepsilon_t}\right)\text{-}0.2\left(\frac{\varepsilon}{\varepsilon_t}\right)^6,\ \ x \le 1 \tag{2.2a}$$

$$\frac{\sigma}{f_t^*} = x \Bigg/ \left[\alpha_t\left(\frac{\varepsilon}{\varepsilon_t}\text{-}1\right)^{1.7} + \frac{\varepsilon}{\varepsilon_t}\right],\ \ x > 1 \tag{2.2b}$$

where α_t=1.25 is the parameter described ascent stage of constitutive model for concrete subjected to tension, f_t^* = 2.0Mpa is the uniaxial nominal tensional strength and ε_t is the strain corresponding to f_t^* = 0.95×10^{-4}.

Constitutive model in accordance to Liu (2005) is used to simulate infill walls. The formula for the model is as follows:

$$\frac{\sigma}{f_m}=1.96\left(\frac{\varepsilon}{\varepsilon_0}\right)-0.96\left(\frac{\varepsilon}{\varepsilon_0}\right)^2,\ \ x\le 1 \tag{2.3a}$$

$$\frac{\sigma}{f_m}=1.2-0.2\left(\frac{\varepsilon}{\varepsilon_0}\right),\ \ 1\le x\le 1.6 \tag{2.3b}$$

where ε_0 is set with 0.003, ε_u is set with 0.0048, f_m is the nominal compressive strength of infill walls. In accordance to GB50003 (2001), nominal compressive strength can be computed by formula $f_m = k_1 f_1^{\alpha}(1+0.07f_2)k_2$, in which k_1 is set with 0.78; α is set with 0.5, k_2 is set with 1, f_1 and f_2 are nominal compressive strength of brisk and mortar, respectively. Nominal tension strength can be computed by formula $f_{tm} = k_3\sqrt{f_2}$, where f_{tm} is nominal tension strength of infill walls and k_3 is set with 0.141.

Damping ratio is assumed to be 5% of the material damping for the general element model. Plane stress element CPS4R is used to model infill walls. Frictional connection is used to model joints between infill walls and frames. The tangential frictional factor is set with 0.7.

3. EVALUATION OF PROGRESSIVE COLLAPSE POTENTIALS

3.1 Incremental dynamic pushdown analysis

Based on load pattern of Alternate Path Method recommended in GSA2003 (2003), and according to analysis method introduced by Khandelwal *et al.* (2008) and Kim *et al.* (2009), incremental dynamic pushdown analysis (IDPA) is applied to investigate the frame resistances. The load α(DL+0.25LL) applied on bays directly above failure column is incrementally increased until frames collapse, while the load (DL+0.25LL) is applied on other bays, in which DL represents the dead load, LL represents the live load and α represents the load factor. The maximum strength less than 1.0 implies that the frames cannot resist the load specified in the GSA2003 (2003), i.e., the service gravity load. Fig. 3.1 shows the loading layout when C2 is removed in incremental dynamic pushdown analysis. To simulate the dynamic phenomenon that the column is suddenly removed, ABAQUS command of *MODEL CHANGE is used. This command can be used to remove elements during analysis. Hence, three analysis steps are existed in the analysis procedure. The first is a static analysis step with gravity analysis, the second is a dynamic analysis step with removal of the column in an appropriate time interval, and the third step is also a dynamic analysis step up to the end of responses. The adopted removal time interval is 1/10*T*, which is the optimization value for the studied frame, where *T* is structural vertical fundamental period. The optimal removal time interval is validated by a numerical verification, which showed that the maximum displacement of joint directly above failure column increase as removed time interval decrease. However, as the removed time interval is shorter than 1/10*T*, the maximum displacement changes little. Hence, 1/10 vertical fundamental period of structures can be used as column removal time interval in progressive collapse analysis for the studied structures.

For simplifying the expression, the column failure case of frame without infill walls is represented by 'WO' and with infill walls is represented by 'W', respectively, e.g., the case of C1 failure of frame without infill walls is represented by 'C1-WO' and with infill walls is represented by 'C1-W'. The pushdown curve illustrated in Fig. 3.3 shows the comparison of load factor between frame without infill walls and infilled frame when the first-story center column C2 is suddenly destroyed. As can be seen, the maximum load factor of infilled frame is larger than the frame without infill walls. According to the result, it is probable that frame with infill walls may achieve higher progressive collapse resistance in this case. The frame without infill walls collapses in C1 removal case, but the infilled frame survives and that load factor is not presented in this case for the limited space. It generally concludes that the infilled frame behaves well in progressive collapse performance.

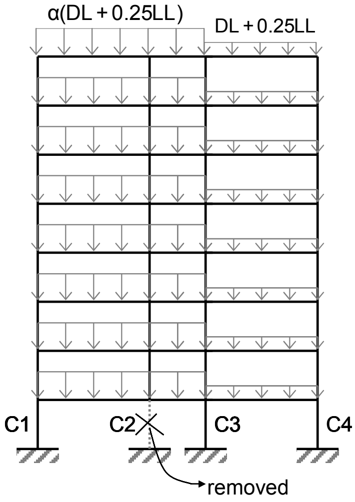


Figure 3.1 Loading layout in incremental dynamic pushdown analysis

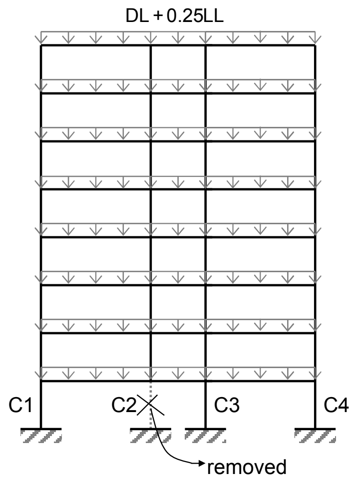


Figure 3.2 Loading layout in nonlinear dynamic analysis

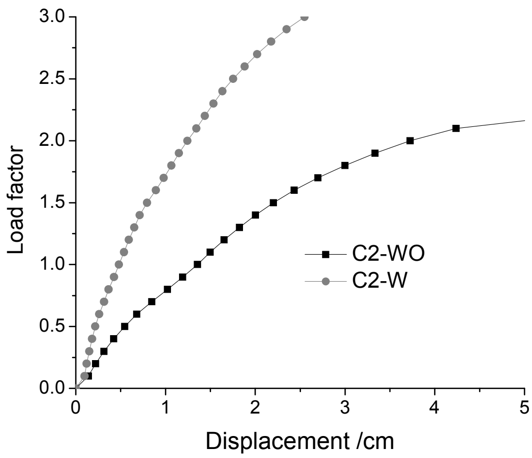


Figure 3.3 IDPA curves of frames without and with infill walls in case C2 removed

3.2 Nonlinear dynamic analysis

Nonlinear dynamic analyses are carried out by suddenly removing the center and the corner columns at the first story, respectively. The load 1.0(DL+0.25LL) is applied on structures as shown in Fig. 3.2 in nonlinear dynamic analyses.

Fig. 3.4 compares the displacement time histories at the top joint of removed column in C1 and C2 removal cases of the frame without infill walls and infilled frame. Since frame without infill walls does not have enough progressive collapse resistance when corner column is removed, its curve is not plotted in the figures. It can be seen that the dynamic response gradually approach to a fixed value, but C1 removal case of frame without walls needs relative long time to reduce. It may be disadvantage to structures for repeating deformation. It also can be seen that displacement is smaller for the infilled frame than frame without walls. It is very probable that infill walls have beneficial effect on progressive collapse resistance of structures. It also can be seen that displacement is smaller in case C2 than case C1 for infilled frames. It is due to cantilever beam mechanism above first story is formed when corner column is removed and it is disadvantaged to load transfer.

Fig. 3.5 compares the axial force time histories of columns in the first story in the C2 removal case of the frame without infill walls and infilled frame. The axial force is normalized by that before C2 removed. It can be seen that C3 in the C2-WO removal case has the highest axial force ratio. It may conclude that C3 is importance to structure in the C2-WO removal case. It can be also seen that axial force of corner column has small fluctuation and axial force of C4 decreases in the C2-WO removal case. However, in C2-W removal case, forces of corner column (C1 and C4) have large fluctuation and center column (C3) change little. It may be due to that infill walls contribute to dissipation of internal force induced by column loss.

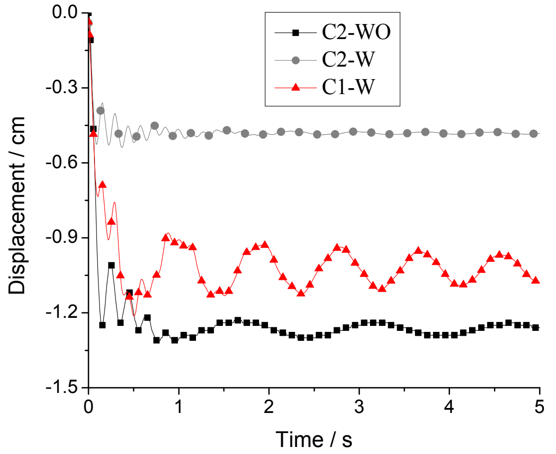


Figure 3.4 Nonlinear dynamic displacement histories

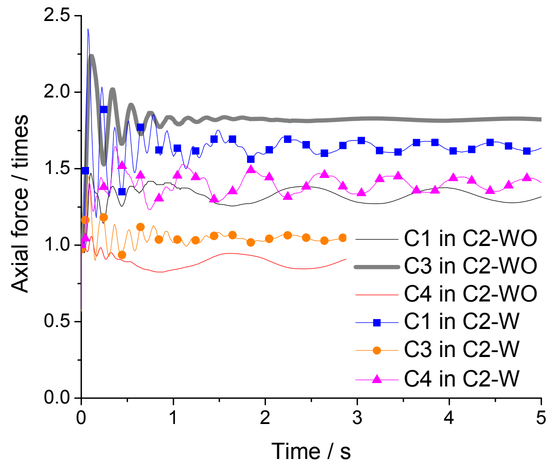
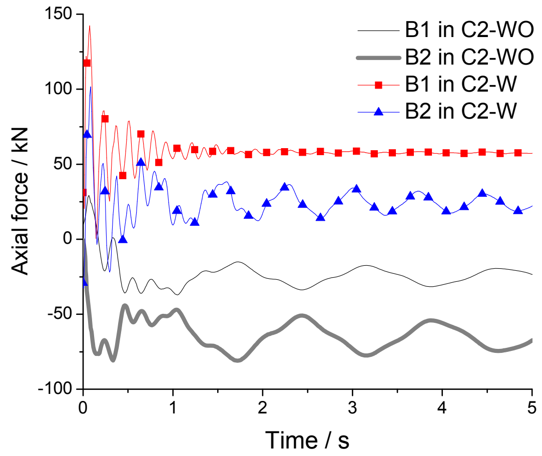
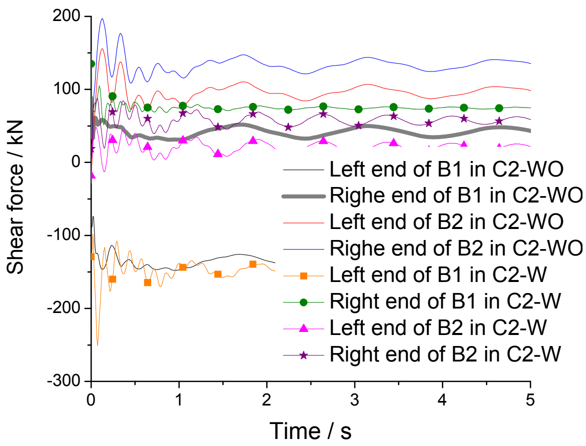


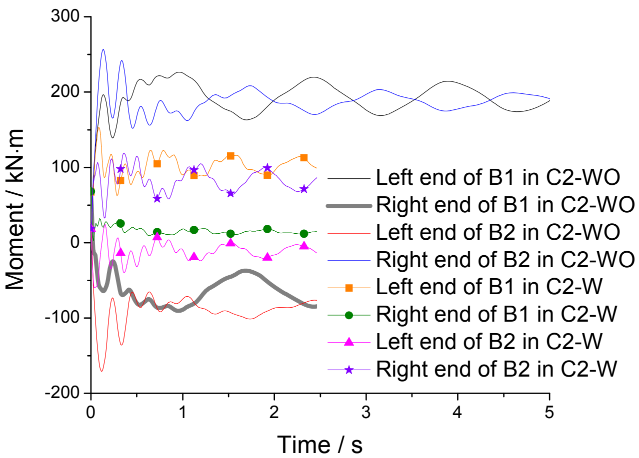
Figure 3.5 Axial force of columns in the first story in case C2-WO and C2-W



(a) Axial force of beams adjacent to failure column in case C2



(b) Shear force of beams adjacent to failure column in case C2



(c) Moment of beams adjacent to failure column in case C2

Figure 3.6 Internal force of beams adjacent to failure column in case C2

Fig. 3.6(a)~(c) provide the internal force time histories of first-story beams in the C2-WO and C2-W removal cases. As shown in Fig. 3.6(a), B1 and B2 subject only to comparison in the C2-WO removal case and subject only to tension in C2-W removal case. It means that infill walls in frame change the load transfer path. As shown in Fig. 3.6(b), shear force on both side of beam B2 in the C2-WO removal case is much larger than that in the C2-W removal case. It can be seen from Fig. 3.6(c) that left side of beam B1 and right side of beam B2 in the case C2-WO have much larger moment than the case C2-W. Those may be due to that internal force in infilled frame dissipates to other members through more alternative paths, such as infill walls, which is beneficial to progressive collapse resistance of frames.

4. EVALUATION OF SEISMIC COLLAPSE POTENTIALS

In order to investigate seismic collapse potentials of the frames, the analytical models are subjected to seismic excitations of CAP090 in 1989 Loma Prieta earthquake, CLW-TR in 1992 Landers earthquake, and LOS270 in 1994 Northridge earthquake. Considering that the examined frames are subjected to strong ground motions for seismic collapse, the maximum acceleration of these excitations are scaled to 0.4g for dynamic analysis according to Chinese code for seismic design of buildings (GB50011, 2010).

Inter-story drift ratio has been widely used as an index to describe structural failure. To evaluate the structural damage, the maximum inter-story drift ratios between the two frames are compared in Fig. 4.1. It can be seen that in the first story, the infilled frame have much higher value compared with that of the frame without infill walls and exceeds 2%, which is larger than limited value specified in GB50011 (2010). It is certain that soft story is formed in the first floor. It also can be seen that maximum inter-story drift ratio of frame without infill walls is comparatively equal in each story and no soft story is formed. Fig. 4.1 also illustrates that inter-story drift ratios are relative less in the infilled frame than those of frame without infill walls in 2 to 8 floors. It is because that infill walls located those floors which increase the lateral stiffness.

In an effort to study the local response of frames, Fig. 4.2 compares time histories of first story displacement of frames. It is shown that first story displacement of infilled frames is much larger than frame without infill walls, where response of the first story of infilled frames vibrates much more deviant from initial location. It further demonstrated that soft first story is formed in the infilled frame. Since infill walls may highly enhance lateral stiffness of a structure, the phenomenon of soft first story is due to the abrupt change of stiffness between first and upper stories.

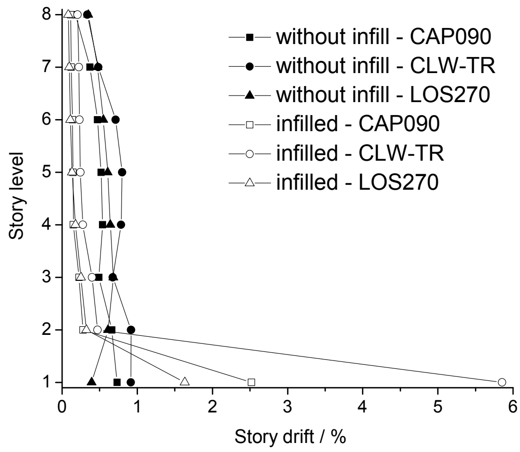
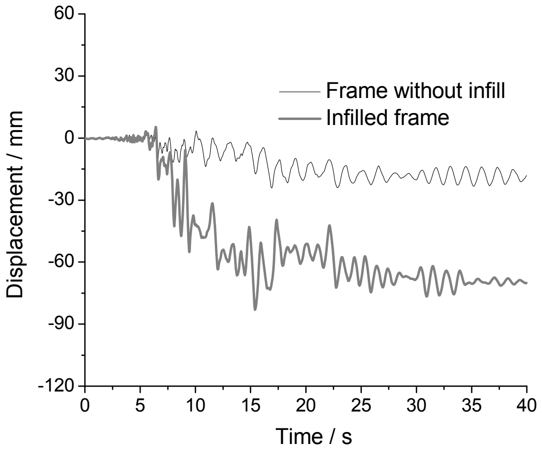
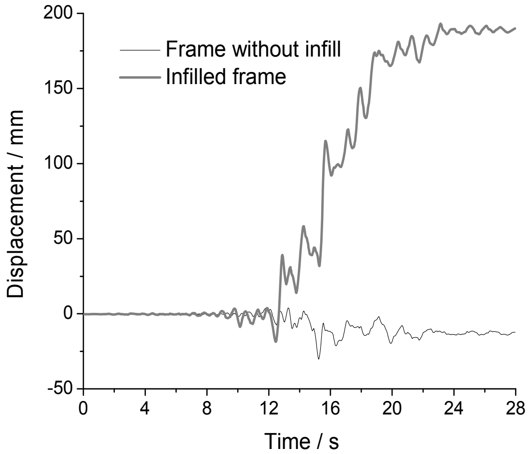


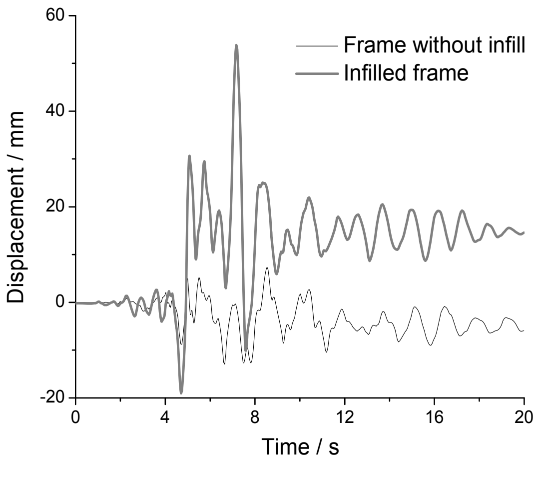
Figure 4.1 Maximum inter-story drift ratio



(a) CAP090



(b) CLW-TR



(c) LOS270

Figure 4.1 Time history of first-story displacement

5. CONCLUSION

The paper highlights the importance of a unified consideration of earthquake loads and accidental loads in the structural collapse resistant analysis and design. The masonry-infill wall effects on structural progressive collapse and seismic collapse are also considered in the analysis. For the selected reinforced concrete frame in the study, frame without infill walls has enough capability to resistant seismic collapse, but fail to resistant progressive collapse. In the other hand, the infill walls increase the resistant capability to the progressive collapse by modification the load transfer path, but forms the soft-story damage under seismic excitations. It is essential to distinguish that the progressive collapse and seismic collapse provide different deformation and force demands on structural columns and beams. In progressive collapse, strong-beam mechanism is better, rather than strong-column and weak-beam mechanism in structural seismic design. The difference of these two type of collapses needs unified analysis when evaluation or design of structures.

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