The 2015 World Congress on
Advances in Structural Engineering and Mechanics (ASEM15)
Incheon, Korea, August 25-29, 2015

Seismic and progressive collapse potentials of low-rise soft-story RC frames and strengthening strategy

*Shuang Li[1)], Xiaomin Wang[2)], Sidi Shan[3)], Changhai Zhai[4)] and Lili Xie[5)]

[1), 2), 3), 4)] *Key Lab of Structures Dynamic Behavior and Control (Harbin Institute of Technology), Ministry of Education, Heilongjiang, Harbin 150090, China*
[1), 2), 3), 4)] *School of Civil Engineering, Harbin Institute of Technology, Heilongjiang, Harbin 150090, China*
[1), 2), 3), 4), 5),] *Institute of Engineering Mechanics, China Earthquake Administration, Heilongjiang, Harbin 150080, China*
[1)] *shuangli@hit.edu.cn*

ABSTRACT

This study investigates the seismic and progressive collapse potentials of frame structures with a soft story in their first stories. Two six-story RC infilled frames designed corresponding to two different seismic intensity levels are considered and their performances on seismic and progressive collapse capacity are analyzed. It is found that the improper design of infilled frames may lead to large diversities among story lateral stiffness, most of plastic deformations developed in first-story columns rather than in beams, which is quite undesirable from seismic resistance standpoint according to the seismic design code; whereas, a beneficial influence on the progressive collapse capacity is usually obtained, although a poor ductility capacity is often accompanied. To improve seismic and progressive collapse performance in a unified procedure, a strengthening strategy by modifying the lateral stiffness ratio of second story to first story of infilled frames is proposed; and rational ratios are discussed. Then the progressive collapse potential of soft-story frames before and after the strengthening is investigated. The finding is that the strengthening strategy can improve the resistance to progressive collapse by increasing the ultimate resistant load factor, and sometimes can increase the ductility capacity.

KEY WORDS: soft-story frame, infill walls, seismic collapse, progressive collapse, strengthening strategy

1. INTRODUCTION

Generally, two major reasons of structural collapses are due to earthquake loads and accidental loads, which are termed as seismic collapse and progressive collapse. In usually designs, however, infill walls are usually treated as non-structural components and only the weight is accounted. In fact, they will interact with the bounding frame and have a significant effect on the performance of a reinforced concrete (RC) frame structure. If infill walls are not properly designed, it may lead to catastrophic structural failures.

Existence of infill walls sometimes changes regularity characteristics of the bare frame and consequently can affect the vibration period of buildings (Chaker and Cherifati 1999; Amanat and Hoque 2006; Crowley and Pinho 2006; Mondal and Tesfamariam 2014), i.e., the dynamic characteristics of the structure, which may result in different seismic actions and failure patterns of a infilled frame structure. A typical failure pattern is characterized by a soft-story collapse mechanism and it has been observed many times in past earthquakes (Fardis and Panagiotakos 1997; Dolšek and Fajfar 2001; Li et al. 2008; Verderame et al. 2011). A soft-story frame is usually referred as a frame with weak stories in its first story due to the absence of infill walls. In fact, this type of frames is quite common because of its economic efficiency and applicability. However, the current seismic design philosophy for frame structures seem as do not contain enough criteria to predict the real response of such buildings suffered from the soft-story collapse mechanism (Alinouri et al. 2013).

In recent years, literatures on progressive collapse have expanded significantly because some extreme collapse events occurred, e.g. the 1995 Alfred P. Murrah Federal Building event in Oklahoma City and 2001 World Trade Center event in New York City. These studies involve the global performances of structures (Kim et al. 2009a; Kim and Lee 2009; Pachenari et al. 2010) and local performances, such as beam catenary action (Khandelwal and El-Tawil 2006), joint effects (Kim et al. 2009b), and slab membrane effect (Alashker 2010). Likewise, the effect of infill walls on progressive collapse is also concerned. Sasani (2008) and Sasani and Sagiroglu (2008) have conducted a field test to investigate the dynamic response of an infilled RC frame in column loss scenarios. Tsai and Huang (2011, 2013) studied the progressive collapse performance of RC frames with different layouts of infill walls. Stinger and Orton (2013) tested a series of two-story RC frames, and they concluded that partial infill walls may not be significant in increasing the progressive collapse resistance of a frame.

Actually, structural collapse resistance evaluation should consider the seismic collapse and the progressive collapse in a unified procedure. However, up to now, such literatures are very limited. Powell (2005) illustrated differences between analyses on seismic collapse and progressive collapse, such as vertical load resistant members, i.e. beam is substantial in progressive collapse, and lateral load resistant members, i.e. column is substantial in seismic collapse. Hayes et al. (2005) analyzed the relationship between seismic detailing and progressive collapse resistance by using the Alfred P. Murrah Federal Building. They found that strengthening the perimeter members using seismic detailing techniques improved the structural survivability, while strengthening the internal members was not nearly as effective in reducing damage. Asprone et al. (2008) performed a probabilistic analysis for a RC frame before and after it is subjected to a seismic retrofit scheme; and it confirms that seismic retrofitting techniques are useful in mitigating the risk of progressive collapse. These findings give an inspiration to find a way to improve the performance of seismic collapse and progressive collapse at the same time. Moreover, the progressive collapse performance of a soft-story frame is different from a bare frame. In some researches (Sasani 2008; Sasani and Sagiroglu 2008; Tsai and Huang 2011, 2013), analyzed structures are possible soft-story structures because there are not infill walls in their first stories. However, this feature is not clearly mentioned in their papers. In addition, these researches only focused on the progressive collapse performances of these structures, the coupled researches together with the seismic performances are not conducted.

This study aims to provide a preliminary insight on the seismic collapse and progressive collapse performances of soft-story RC frames, and the effects of strengthening strategy on seismic and progressive collapse resistant capacity. The paper is organized as follows. The finite element modeling approach is presented in Section 2. Then, the seismic and progressive collapse potentials are studied in Section 3. The evaluation of strengthening strategy on seismic and progressive collapse potentials is provided in Section 4. Some concluding remarks complete the study.

2. ANALYSIS MODELING APPROACH

In the past few decades, finding an accurate and rational analytical model to evaluate the response of the RC frame structure with infill walls has always been an important issue. Polyakov (1960) conducted a series of experiments on infilled frames, and first proposed the concept of using equivalent compression diagonal strut to represent the infill wall. Holmes (1963) proposed that the effective width of an equivalent strut depends primarily on the thickness and the aspect ratio of the infill wall, and suggested that one-third of the infill diagonal length can be taken as the effective width of the equivalent strut. Since then, various strut models including single strut models (Saneinejad and Hobbs 1995; Kadysiewski and Mosalam 2009) and multiple strut models (Chrysotomou 1991; El-Dakhakhni et al. 2003; Fiore et al. 2012) were proposed and they have been widely used in the analysis of the infilled frame structures.

In this study, a modified three-strut model is used to simulate the behavior of the infilled frames (Saneinejad and Hobbs 1995; El-Dakhakhni et al. 2003; Zhai et al. 2011). The geometry model for the modified three-strut model is shown in Fig. 1, in which the bounding frame is modeled by beam-column element and struts are modeled by truss elements. The modification of the model is mainly based on the following considerations: (1) The contact length and the equivalent diagonal strut width are not constant but varying with the applied load. Flanagan and Bennett (1999) has shown that the equivalent strut width at ultimate state is approximately one-half of the strut width for the initial state; and the numerical simulations by the authors verify this point too. Consequently, the total width of three struts is increased to 1.5 times of the equivalent strut width at the ultimate state; and (2) In the original three-strut model, it is assumed that the contact stress distribution along the contact areas is uniform, which derives that the central strut width is one-half of the total width of three struts at the ultimate state and the widths of two side struts are both equal to a quarter of the total width. However, the fact is not true because the contact stress distribution is more like a parabolic distribution along the contact areas. These all imply that the central strut should resist more applied load. As a result, the central strut width takes 2/3 of the total width to play a more important role in resisting the applied load; the widths of the two side struts are 1/6 of the total width, respectively.

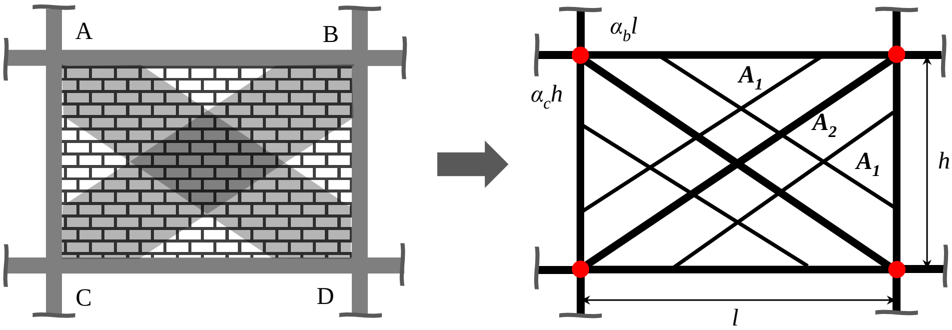


Fig. 1. The three-strut model for infilled frame.

To verify the analysis model introduced above, three one-story one-bay RC frame specimens are simulated. These specimens were tested by Mehrabi and Shing (1997) which were numbered with specimen 1, 6 and 7 respectively in their tests. Specimen 1 is a bare frame, specimen 6 and 7 are two infilled frames with different masonry types. The analysis is performed by OpenSees (Mazzoni et al. 2009). Beams and columns are modeled using the nonlinear beam-column element with distributed plasticity along the beam length and fiber discretization within the cross-section. The section force-deformation relationships are calculated by integrating over all the fibers in the cross-section. In this case, 144 fibers for the core concrete, 112 fibers for the cover concrete, and a fiber per each reinforcing bar are defined for the different sections of the beams and column elements. Material properties are then assigned to each fiber. The concrete material is defined with the model of Concrete02 uniaxial stress-strain relationship. And the reinforcing bar material is defined using the model of Steel02 uniaxial stress-strain relationship. For six struts, they are modeled using the truss element with only compressive stress to resistant. Hence, the model of Concrete01 with no tensile stregth is adopted.

The experimental and numerical results of lateral load-displacement relationship for specimen 1, 6 and 7 are plotted in Fig. 2. Fig. 2(a) shows the lateral load-displacement curves of specimen 1, which illustrates that using the nonlinear beam-column element in Opensees to simulate the columns and beams can provide sufficient accuracy to model the behavior of the bare frame. Fig. 2(b) and (c) give a comparison between the numerical and experimental results of specimen 6 and 7, show that the numerical results obtained with the modified three-strut model have an excellent agreement with the experimental results. It is applicable to use the modified three-strut model to model the infilled frame.

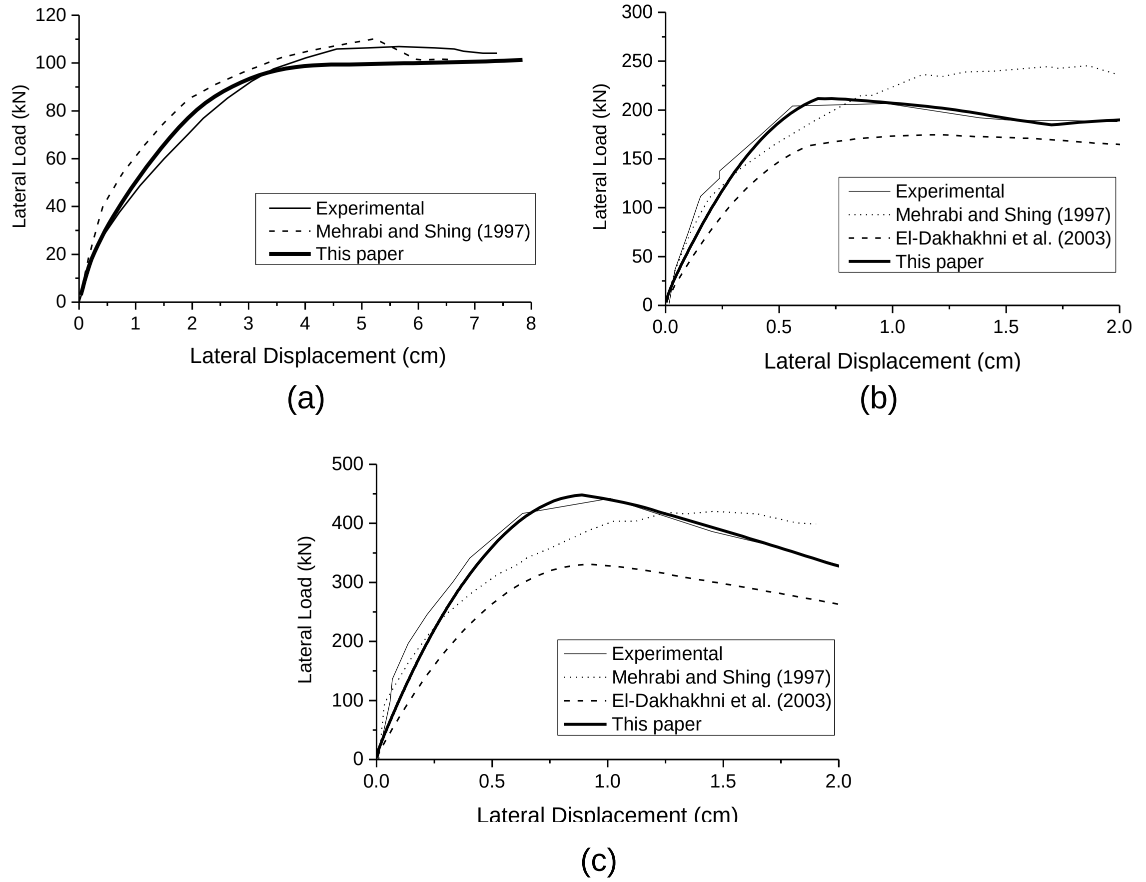


Fig. 2. Lateral load-displacement curves for (a) specimen 1, (b) specimen 6, and (c) specimen 7 in Mehrabi and Shing (1997) test.

3. SEISMIC AND PROGRESSIVE COLLAPSE POTENTIALS

3.1 Case study structures

In this study, a typical soft-story frame is chosen as the prototype structure to evaluate its seismic and progressive collapse potentials. This kind of frame structures can be found almost everywhere in China; and for various purposes, such as school, hospital, dormitory, apartment and so on. In 2008 Wenchuan earthquake and 2010 Yushu earthquake, there are lots of these structures found collapsed. Therefore, case study structures used here are designed in accordance with Chinese seismic code (GB50011 2001), and two six-story RC frames with infill walls in level 7 and 8 of seismic intensity regions are built. Their site classifications both belong to Site II. These two structures have the same structural configuration and same distribution of the infill walls, which are shown in Fig. 3. The structural configuration is regular, whereas the vertical distribution of the infill walls is irregular due to architectural needs and it induces a soft-story in the first story. In the following, these two frames in level 7 and 8 of seismic intensity regions are called FRAME1 and FRAME2, respectively. For evaluating the influence of the infill walls on the collapse performance of these frames with soft stories, the frame models with and without the infill walls are both analyzed.

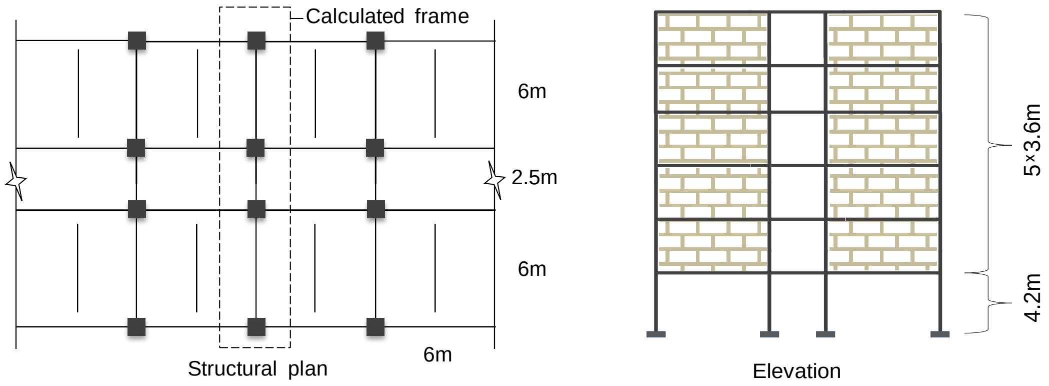


Fig. 3. Structural configuration.

Table 1 and Table 2 give the dimension and rebar of columns and beams for these two RC frames, respectively. The compressive yield strength of concrete of columns and beams is 30 MPa, the yield strength of longitudinal rebar is 335 Mpa, and the hooping yield strength is 235 MPa. The infill wall has the thickness of 240 mm and the compressive strength of 7.8 MPa.

Table 1. Design information of FRAME1.

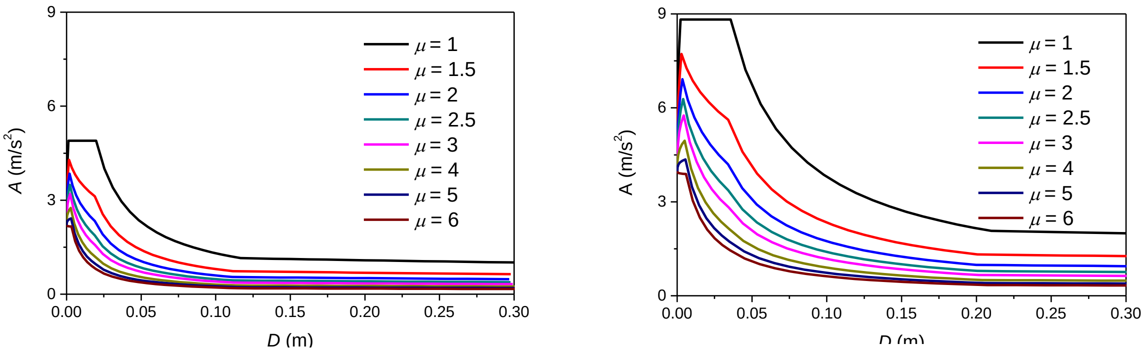
Story	Column (mm×mm)	Rebar of Column at Each Side (mm^2)	Beam (mm)		Rebar of Beam mm^2			
			Internal	External	Internal		External	
			Width: 250 Height		Top	Bottom	Top	Bottom
1	450×450	1205	500	400	1570	1256	1570	1256
2	450×450	1205	500	400	1570	1256	1570	1256
3	450×450	1018	500	400	1570	1256	1570	1256
4	450×450	1018	500	400	1570	1256	1570	1256
5	400×400	764	500	400	1256	1017	1256	1017
6	400×400	764	500	400	1256	1017	1256	1017

Table 2. Design information of FRAME2.

Story	Column (mm×mm)	Rebar of Column at Each Side (mm^2)	Beam (mm) Internal	Beam (mm) External	Rebar of Beam (mm^2) Internal Top	Internal Bottom	External Top	External Bottom
			Width: 250 Height					
1	500×500	2499	550	450	2202	1884	2281	1884
2	500×500	2499	550	450	2202	1884	2281	1884
3	500×500	1900	550	450	2202	1884	2281	1884
4	500×500	1900	550	450	2202	1884	2281	1884
5	450×450	1706	550	450	1520	1256	1520	1256
6	450×450	1706	550	450	1520	1256	1520	1256

3.2 Seismic collapse performance analysis results

Seismic performances of two frame structures with and without the effect of infill walls are investigated by the pushover analysis. For the displacement response solution, the improved capacity-demand diagram method that uses the constant-ductility spectrum for the seismic demand curve (Chopra and Goel 1999) is used. The seismic demand curve is based on reduction of elastic design spectrum in Chinese seismic code (GB50011 2001) by the yield strength reduction factor R (Vidic et al. 1994). According to Chinese seismic code, the peak values of the elastic design spectra are 0.5g and 0.9g (corresponding to level 7 and 8 of seismic intensity regions, a 2-3% probability of being exceeded during 50 years) for FRAME1 and FRAME2, respectively. Finally, seismic demand curves derived by seismic design spectra for the two frames are presented in Fig. 4.



(a) (b)

Fig. 4. Demand curves derived by design spectrum (a) For FRAME1, (b) For FRAME2.

Distributions of story drift ratio of FRAME1 and FRAME2 are shown in Fig. 5. The observation is that the infill walls significantly change the distribution of the story drift ratio. For bare frames, the story drift ratio of each story varies slightly, and relative large values occur at the second and third stories. Whereas, for the infilled frames, infill walls strengthen the upper stories, which induce the deformation of the first story rapidly increases and upper story deformation decreases markedly, and a soft story in the first floor has formed. Therefore, for a frame structure with a soft story in its first story, it may over-strengthen the upper stories and induce catastrophic failures in columns of the first story, which is undesirable from seismic resistance standpoint. It is worth to note that although infill walls change story drift ratios, the maximum story drift ratios of the two frames still satisfy the limit value (2% for frame structure) to judge collapse by the seismic design code (GB50011 2001).

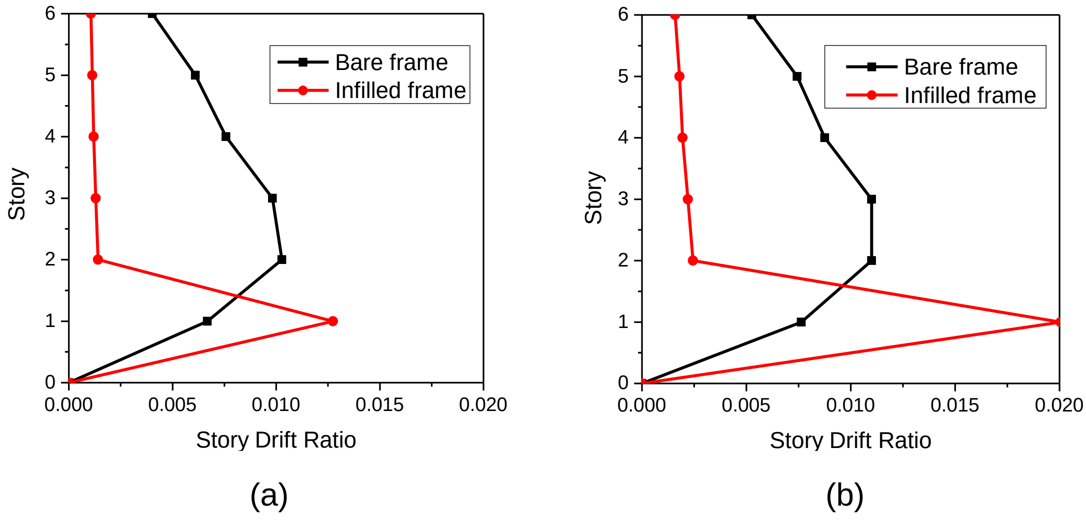
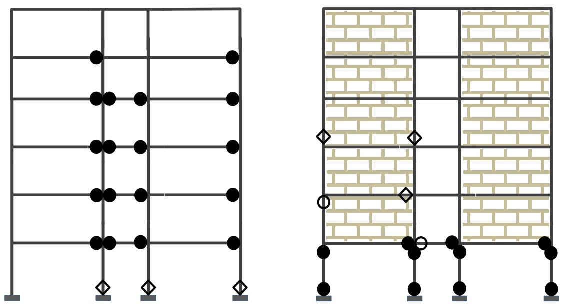


Fig. 5. Comparison of story drift ratios (a) For FRAME1, (b) For FRAME 2.

Next, distributions of the plasticity development in columns and beams of FRAME1 and FRAME2 are evaluated and they are shown in Fig. 6 and Fig. 7, respectively. Clearly, these two Fig.s illustrate that, when infill walls are not considered, most of the plasticity developments occur in beams, and the seismic design principle of strong-column weak-beam failure mechanism realized. In the case of the infilled frames, all the columns in the first story have severe plasticity developments and a soft-story failure mechanism forms. The analysis results agree well with the observation in practical earthquakes, where seismic performance of infilled frames may be insufficient if infill walls are not well considered in design. The results also illustrated that under the considered rare earthquake excitations in the design earthquake scenarios of the two frames, the frames don't reach their collapse limits but serious plastic deformations in columns are developed.

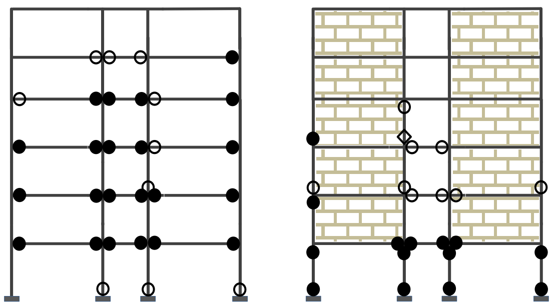
3.3 Progressive collapse performance analysis results

Nonlinear static and dynamic analysis methods are applied to investigate progressive collapse potentials. These two methods are recommended in GSA (2003) and DoD (2010). The nonlinear static analysis, i.e., pushdown analysis is conducted by gradually increasing the vertical displacement at the location of the column removed point to investigate the resistant capacity against such deformation. The load combination of 1(Dead Load + 0.25×Live Load) is used here; and the load is applied uniformly in entire spans (Khandelwal and El-Tawil 2008), as shown in Fig. 8(a) and (b). The external column and internal column in the first story are removed as two scenarios in the analysis. The applied load pattern remains unchanged during the analysis. Whereas, corresponding loads to displacement levels of the location of the removed column are determined after each displacement incremental step. To reflect the progressive collapse resistant capacity, a load factor is defined as the ratio of corresponding loads to 1(Dead Load + 0.25×Live Load). It should be noted that the load combination used for the pushdown analysis is only a load pattern, i.e., loads proportionally applied in this pattern but load values varied with the control displacement.



(a) (b)

Fig. 6. The plasticity development of FRAME1 (a) without the infills, (b) with the infills.



(a) (b)

Fig. 7. The plasticity development of FRAME2 (a) without the infills, (b) with the infills.

Note: ● Plasticity, ○ Small plasticity just occur, ◇plasticity almost occur

Relationships between the load factor and vertical displacement of the removed point for FRAME1 are plotted in Fig. 9. Both bare frame and infilled frame are considered. It is observed that infill walls increase yielding strength of frame structures. For the case of external column removed, an increase of about 4 times is achieved compared with the bare frame, as shown in Fig. 9(a). The bare frame can’t resistant the progressive collapse after column removed since the maximum load factor is much smaller than 1. Fig. 9(b) presents the case of internal column removed, and a increase of about 1.4 times is obtained. Compared to the exteranl column removed case, it illustrate larger maximum load factors. It should be mentioned that the maximum load factor of bare frame is larger than 1 which means there is a big chance that the bare frame will also survive. In the sense, infill walls indeed improve the progressive collapse resistant capacity, although they may lead to a soft-story collapse subject to the horizontal earthquake loads. However, it is also shown that the ductility capacity of the infilled frame decreases compared with the bare frame, which implies a brittle collapse mode. Similarly, load-displacement curves of the column remove point for FRAME2 are shown in Fig. 10. It can be observed that this figure illustrates alike characteristics to Fig. 9. Only difference is that the load factors are larger.

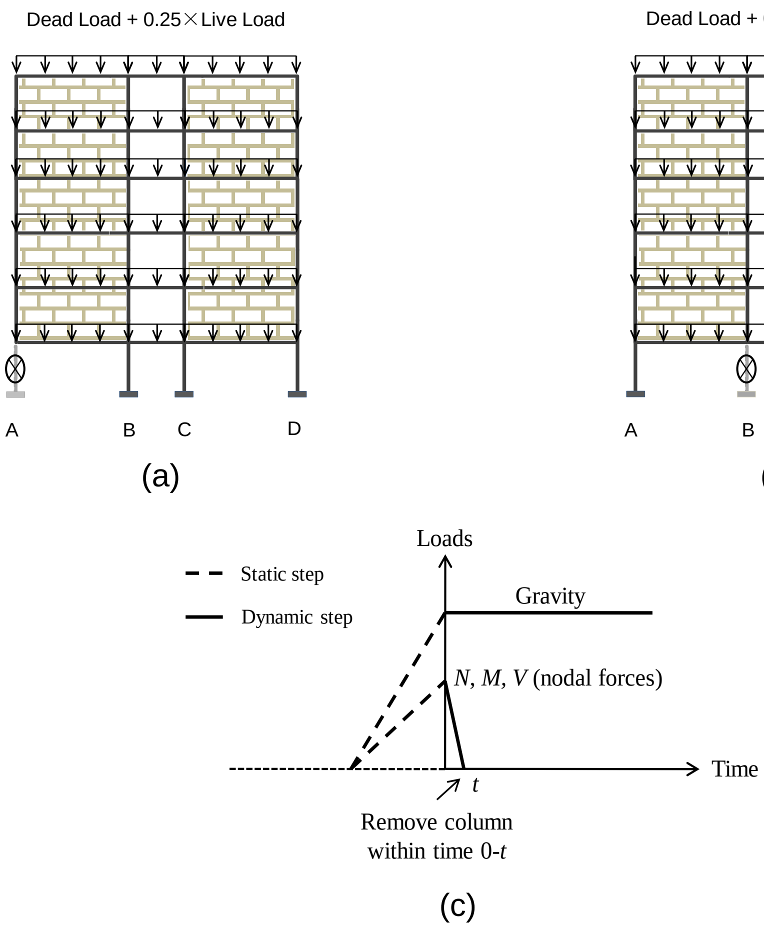
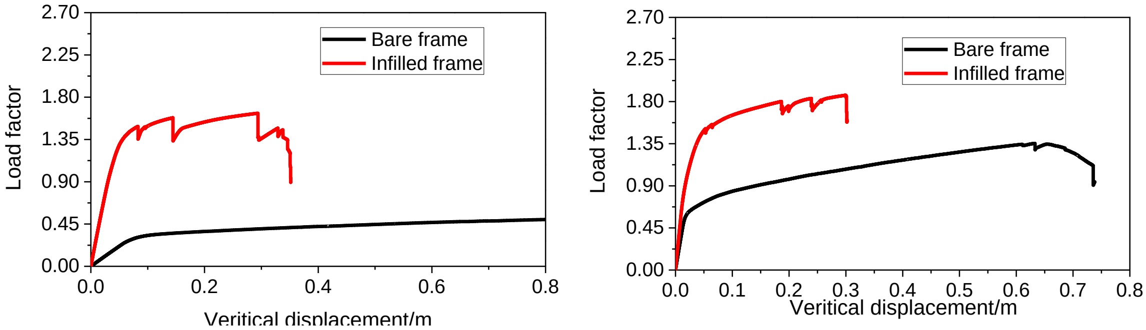


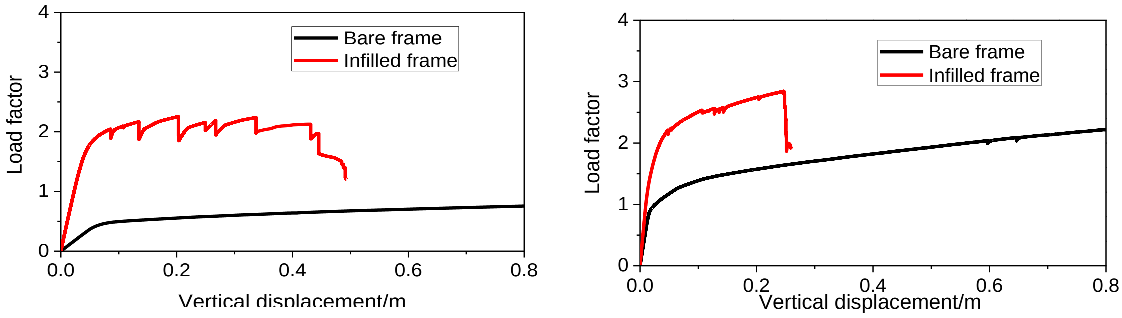
Fig. 8. Applied loads in static and dynamic analyses (a) With external column removed, (b) With internal column removed, (c) Dynamic analysis.

In the nonlinear time-history analysis, the load combination 1(Dead Load + 0.25×Live Load) is also used (see Fig. 8(a) and (b)). For dynamic analyses, the full structural model is first analyzed subject to gravity load, and the internal nodal forces at the upper point (i.e., the column remove point) are obtained. A new structural model is then remodeled without the removed column. The internal nodal forces are applied gradually to the column remove point together with the gravity load. The loads applied procedure is shown in Fig. 8(c). The applied nodal forces are decrease to zero at a very small time interval (0.005s is used here, which is smaller than 1/10 of the first vertical natural period of the structure with the column removed) in the dynamic analysis step. In this way, the progressive collapse analyses start when structures are already deformed by gravity load, and the dynamic effects in the progressive collapse process are considered. The Rayleigh damping with 5% damping ratio is used.



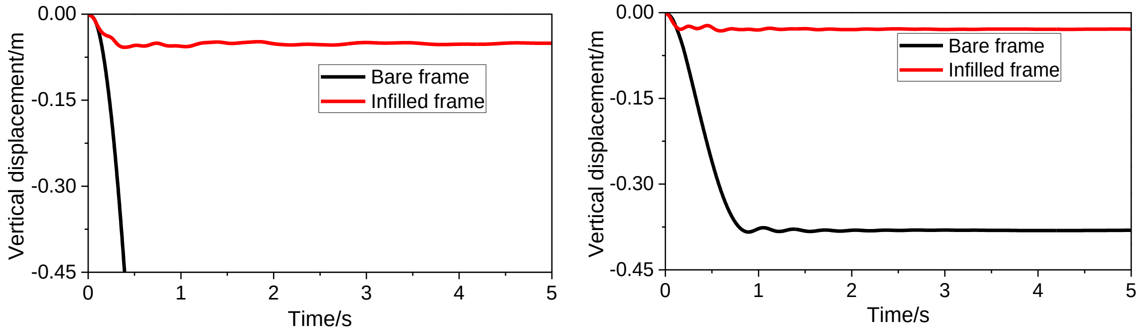
(a) (b)

Fig. 9. Pushdown curves of FRAME1 with (a) the external column removed and (b) the external column removed.



(a) (b)

Fig. 10. Pushdown curves of FRAME2 with (a) the external column removed and (b) the external column removed.

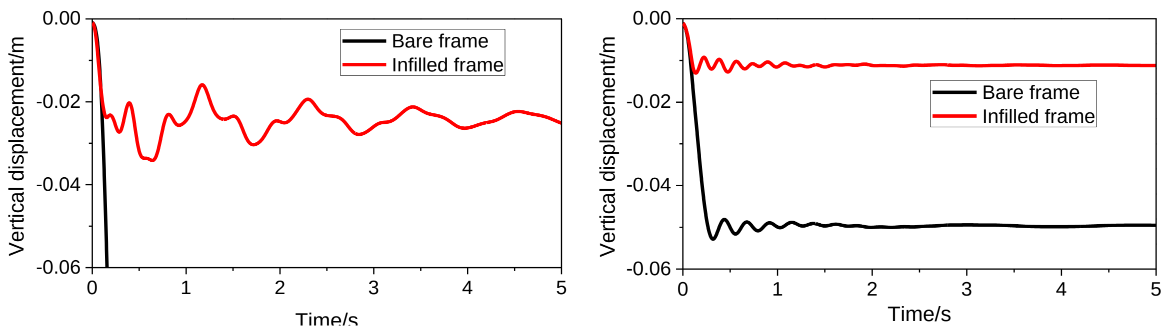


(a) (b)

Fig. 11. Displacement time-history of FRAME1 with (a) the external column removed and (b) the external column removed.

The vertical displacement time-history results of the column removed point for FRAME1 caused by sudden loss of the external and internal column are shown in Fig. 11. The bare frame collapses when the external column is removed, i.e., as soon as the external column is removed, other members failure followed and structure failed without oscillation under 1(Dead Load + 0.25×Live Load). The infilled frame survived although there is a vertical displacement about 0.06 m. For the internal column removed case, a significant observation is that the bare frame survived, although the vertical displacement has reached 0.4 m.

Fig. 12 shows the vertical displacement time-history results of FRAME2. This figure illustrates similar characteristics of FRAME1 as shown from Fig. 11. FRAME2 is designed in the region with higher seismic intensity level than FRAME1, which leads to the displacements of FRAME2 are smaller than the corresponding cases of FRAME1. It is also observed that the displacement oscillation responses of FRAME2 are more obvious than FRAME1. This is because the small displacements make more elastic responses in FRAME2, while the large displacements make more plastic responses in FRAME1.



(a) (b)

Fig. 12. Displacement time-history of FRAME2 with (a) the external column removed and (b) the external column removed.

4. STRENGTHEN STRATEGY

In this section, a preliminary strengthen strategy is suggested to improve both the seismic and progressive collapse capacity. Based on the above results, it can be concluded that, infill walls may induce severe plasticity developments in columns which is quite undesirable for the seismic collapse performance; and a seismic design refinement is urgently needed. Meanwhile, infill walls are beneficial to the progressive collapse performance, although they will decrease the ductility capacity. In actual design, these two collapses should be considered in a unified framework. For progressive collapse, beam strengthening is a direct way to improve its performance; but, it is not a proper option here because it may violate the seismic design principle of strong-column weak-beam failure mechanism. For seismic collapse, it has been mentioned that seismic strengthening may also improve the structural resistance to progressive collapse. Therefore, a proper seismic strengthening is the priority and its effectiveness is verified.

Generally, smooth of the distributions of lateral stiffness in structures is an effective approach to strengthen its seismic performance. In this case, there are several ways to realize this point, such as enlarging the cross-section size of bottom columns, changing the thickness of infill walls (e.g., masonry brick with different sizes) and changing the material of infill walls (e.g., masonry brick with different materials) to the initial design of frame structures. An important issue is how to quantitatively evaluate distributions of lateral stiffness in structures. For a soft story frame, the ratio of initial lateral stiffness in two bottom stories seems to be a simple and good index to reflect the structural performance. The i^{th}-story lateral stiffness of $K_{s,i}$ can be calculated using the following formula:

$$K_{S,i} = \begin{cases} K_{f,1} & i=1 \\ K_{f,2}+K_{w,2} & i=2 \end{cases} \qquad \text{Eq. (1)}$$

where $K_{f,1}$ and $K_{f,2}$ is the stiffness contributed by bare frame. $K_{w,2}$ is the stiffness contributed by infill walls, which can be calculated with the expression as

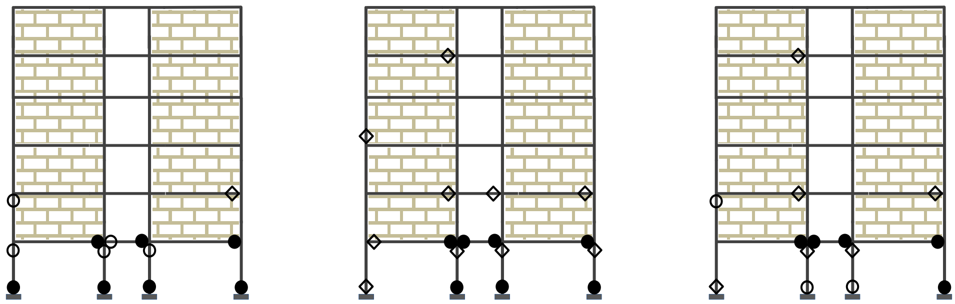
$$K_{w,2} = \sum_j K_{w,2,j} = \sum_j \frac{1}{\dfrac{H_w^3}{3E_w I_w} + \dfrac{3H_w}{E_w A_w}} \quad (j=1,2,3,\cdots) \qquad \text{Eq. (2)}$$

where $K_{w,2,j}$ is the stiffness contributed by j^{th} infill wall. H_w is the height of j^{th} infill wall. E_w is the material modulus of j^{th} infill wall. I_w and A_w is the moment of inertia and the area of the j^{th} infill wall cross-section, respectively.

4.1 Seismic collapse potentials

A series of infilled frames with different ratios of lateral stiffness in two bottom stories are analyzed to examine the effectiveness of $K_{S,2}/K_{S,1}$ ratios. For FRAME1 case, ratios of 12.25(initial frame), 9.18, 7.08, 6.68, 5.01, 3.87, 3.33, 2.96, 2.95, and 2.78 are analyzed; for FRAME2 case, ratios of 8.66(initial frame), 6.67, 5.26, 4.87, 3.75, 2.95, 2.56, 2.54, and 2.18 are analyzed. They are all realized by changing the cross-section size of bottom columns, the thickness of infill walls and the material of infill walls. The distributions of plasticity developments for all these cases are investigated to estimate the structural seismic performance. The rational ratios of lateral stiffness in two bottom stories for FRAME1 and FRAME2 are then discussed.

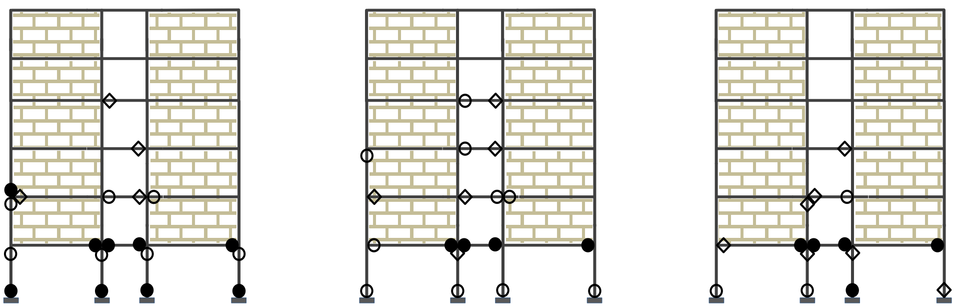
For simplicity, only results of three infilled frames with critical stiffness ratios of 3.33, 2.96 and 2.95 are given for FRAME1 case. The distributions of plasticity development of three frames are shown in Fig. 13. It illustrates that for $K_{S,2}/K_{S,1}$ = 3.33, the plasticity developments occur severely on all the column bases and slightly on part of the column tops, so the strong-column weak-beam failure mechanism is not realized; while $K_{S,2}/K_{S,1}$ is 2.96 and 2.95, the plasticity developments occur slightly on the column bases and the more occur on the beam ends, it can be considered that the strong-column weak-beam failure mechanism is basically realized. It should be noticed that, in Fig. 13(b) and (c), these two cases with the $K_{S,2}/K_{S,1}$ of 2.96 and 2.95 are obtain through different ways (combination of different column size, masonry brick materials and different masonry brick sizes, i.e., the former with weaker infill walls and the latter with stronger column). The distribution of plasticity developments of them are similar, it indicates that the stiffness ratio is a reasonable index to realize the strong-column weak-beam failure mechanism.



(a) (b) (c)

Fig. 13. The plasticity development of FRAME1 (a) $K_{S,2}/K_{S,1}$=3.33, (b) $K_{S,2}/K_{S,1}$=2.96, (c) $K_{S,2}/K_{S,1}$=2.95.

Similar to FRAME1 case, only results of three frames with critical stiffness ratios of 2.95, 2.56 and 2.54 are given for FRAME2 case. Fig. 14 shows the distributions of plasticity developments of each frame, it is considered that when $K_{S,2}/K_{S,1}$ = 2.95, the strong-column weak-beam failure mechanism fails to realize; while $K_{S,2}/K_{S,1}$ is 2.56 and 2.54, the strong column weak beam failure mechanism basically realized. These results illustrated that if $K_{S,2}/K_{S,1} \leq 3$ for the infilled FRAME1 and $K_{S,2}/K_{S,1} \leq 2.5$ for the infilled FRAME2, the two frames have better performances under the earthquake excitations. In the design of an infilled frame, these rational $K_{S,2}/K_{S,1}$ values are desired more systemic research in future.



(a) (b) (c)

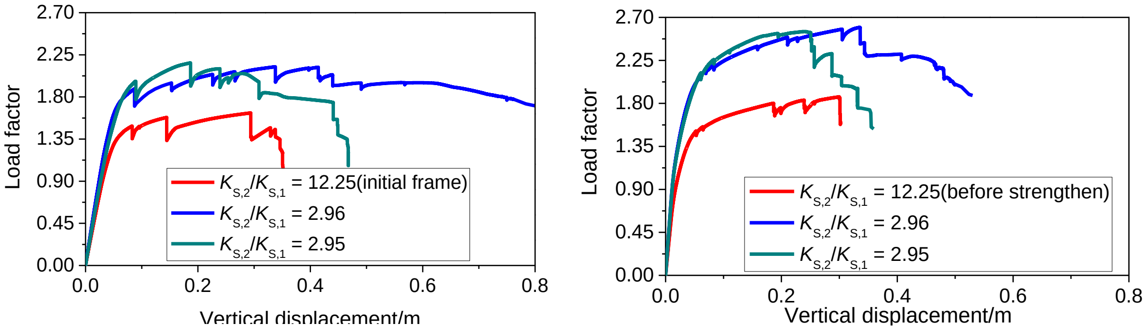
Fig. 14. The plasticity development of FRAME2 (a) $K_{S,2}/K_{S,1}$=2.95, (b) $K_{S,2}/K_{S,1}$=2.56, (c) $K_{S,2}/K_{S,1}$=2.54.

4.2 Progressive collapse potentials

Pushdown analysis results for FRAME1 after strengthening are shown in Fig. 15.

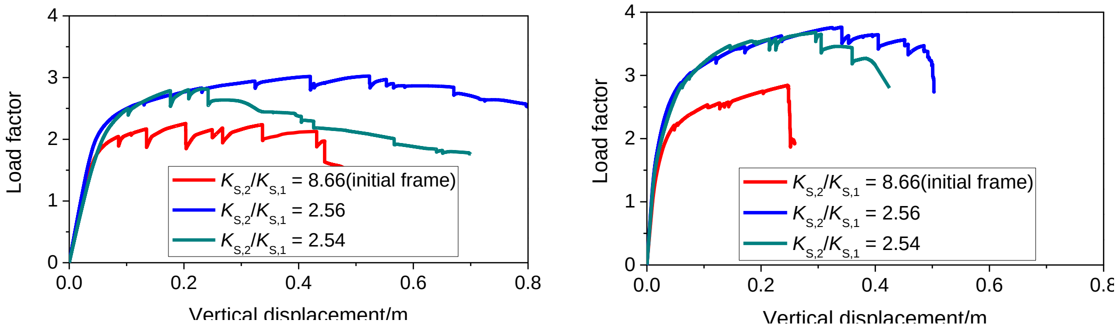
Because the case of $K_{S,2}/K_{S,1}$ = 3.33 did not satisfy the seismic strengthening demand, its result is not plotted in the figure, and only the cases of $K_{S,2}/K_{S,1}$ = 2.96 and $K_{S,2}/K_{S,1}$ = 2.95 are given. It can be seen that, after the seismic strengthening, the maximum load factor is enlarged and ductility capacity is also improved. The cases of $K_{S,2}/K_{S,1}$ = 2.96 and $K_{S,2}/K_{S,1}$ = 2.95 have similar maximum load factors, and about 1.4 times increase achieved. It is worth to note that the load factor is a relative value rather than an absolute value. Therefore, a larger load factor may not correspond to larger vertical loads because weights of themselves may be smaller since different masonry material and infill wall width are used, i.e., the load factor illustrates a comprehensive effect of both the weight and strength of infill walls. Another observation is the case of $K_{S,2}/K_{S,1}$ = 2.96 and $K_{S,2}/K_{S,1}$ = 2.95 have a better ductility capacity. Particularly, the case of $K_{S,2}/K_{S,1}$ = 2.96 provides the best ductility capacity among the infilled frames, although still smaller than the bare frame. In general, the frame with thinner infill walls will provide better ductility capacity since it is more close to the bare frame. However, for the analysis cases, the case of $K_{S,2}/K_{S,1}$ = 2.96 has stronger columns in the first story than the case of $K_{S,2}/K_{S,1}$ = 2.95. Hence, large deformations (displacement and rotation) at the upper point of first-story column in axis B (see Fig. 8(a)) are found for the case of $K_{S,2}/K_{S,1}$ = 2.95, which decreases its ductility capacity. From the Fig. 15, two conclusions can be reached. One is that, compared with the original infilled frame, a rational seismic deign of infill walls can well improve the structural progressive collapse resistant capacity in both load carrying strength and ductility. The other one is that the column, which adjusted to the removed column, may undergo large deformation and play more important roles on the total structural ductility.

Similarly, load factor-displacement curves of the column remove point for FRAME2 are shown in Fig. 16. Only the two strengthen cases of $K_{S,2}/K_{S,1}$ = 2.56 and $K_{S,2}/K_{S,1}$ = 2.54 are plotted here. It can be observed that this figure illustrate alike characteristics to Fig. 15. However, the amplitudes of load factors are different. When the external column is removed, the maximum load factors of FRAME2 bare frame, infilled frame with $K_{S,2}/K_{S,1}$ = 8.66, infilled frame with $K_{S,2}/K_{S,1}$ = 2.56, and infilled frame with $K_{S,2}/K_{S,1}$ = 2.54 are 0.9, 2.26, 3.03 and 2.82; while the maximum load factors of FRAME1 bare frame, infilled frame with $K_{S,2}/K_{S,1}$ = 12.55, infilled frame with $K_{S,2}/K_{S,1}$ = 2.56, and infilled frame with $K_{S,2}/K_{S,1}$ = 2.54 are 0.54, 1.55, 2.12 and 2.16. Therefore, the amplification factors are 1.67, 1.46, 1.43 and 1.31, respectively. When the internal column is removed, the maximum load factors of FRAME2 are 2.22, 2.84, 3.76, and 3.68; while the maximum load factors of FRAME1 are 1.35, 1.87, 2.6 and 2.55. The amplification factors are 1.64, 1.52, 1.45 and 1.44, respectively. Remember that FRAME2 is designed in the region with higher seismic intensity level than FRAME1. Hence, the structure designed in high seismic intensity regions have larger progressive collapse resistant capacity than the structure designed in low seismic intensity regions. For a quantitative estimation, a more than 30% increase to the progressive collapse capacity is achieved when the structural design from level 7 to level 8 of seismic intensity. Moreover, it can be concluded that, when the external column is removed, the progressive collapse capacity of rational designed infilled frame is increased 3~4 times compared with its corresponding bare frame; and 1~2 times increase achieved when the internal column is removed.



(a) (b)

Fig. 15. Pushdown curves of FRAME1 with (a) the external column removed and (b) the external column removed after seismic strengthening.



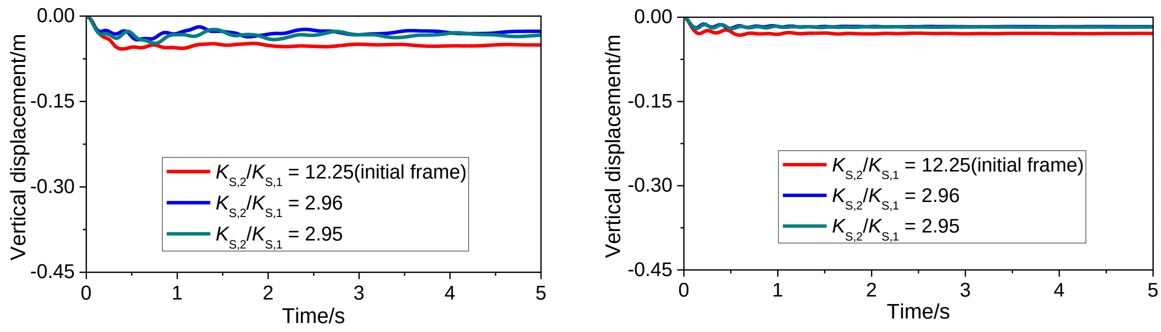
(a) (b)

Fig. 16. Pushdown curves of FRAME2 with (a) the external column removed and (b) the external column removed after seismic strengthening.

In the following, nonlinear time-history analyses are also conducted. In comparison with the frame with $K_{S,2}/K_{S,1}$ = 12.25(initial frame), frames after seismic strengthening (cases with $K_{S,2}/K_{S,1}$ =2.96 and $K_{S,2}/K_{S,1}$ =2.95, shown in Fig. 17) provide smaller vertical displacements in the progressive collapse process. This result implies that the structural seismic strengthening indeed benefits the resistance of progressive collapse. It can be observed that the vertical displacement of the case with $K_{S,2}/K_{S,1}$ =2.96 is similar as the case with $K_{S,2}/K_{S,1}$ =2.95. Fig. 18 show the vertical displacement time-history results of FRAME2 after seismic strengthening. This figure illustrates similar characteristics of FRAME1 as shown from Fig. 17. In these cases, the seismic strengthening indeed decreases the vertical displacements in the progressive collapse process.

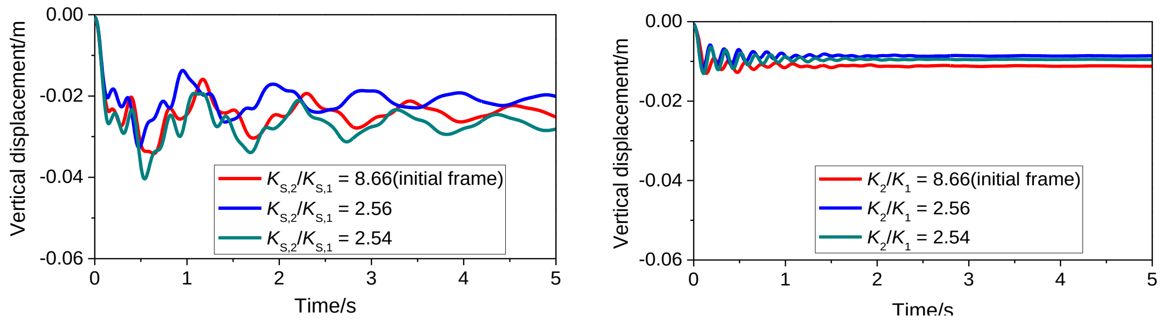
4.3 Analysis results with earthquake-induced damages

In fact, according to seismic codes, structures are designed to experience plasticity development to some extent during strong earthquakes. After an earthquake happened, other damage actions such as earthquake-induced fire or gas blasting usually follows, which may cause further failures on the structures, such as progressive collapse. The structures may have damages or may have residual deformations when subjected to the earthquake excitations, which can also decrease the structural performance. Therefore, it is worth to verify the structural progressive collapse resistant capacity after the earthquake excitation.



(a) (b)

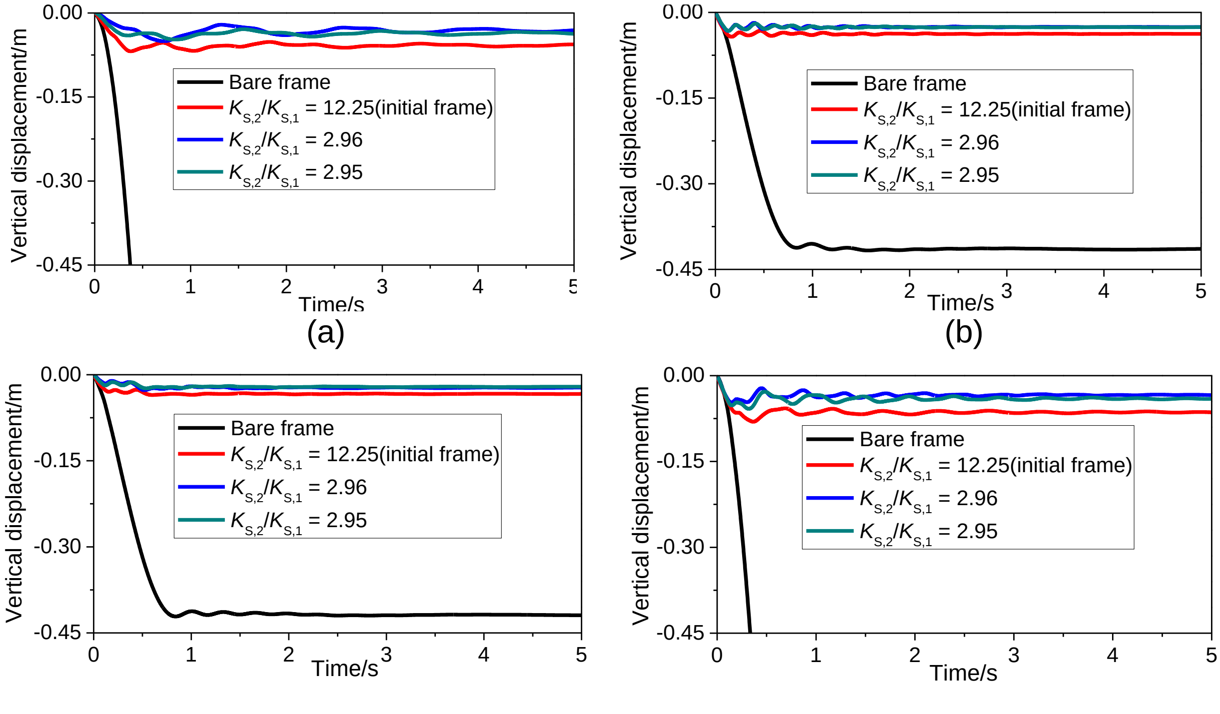
Fig. 17. Displacement time-history of FRAME1 with (a) the external column removed and (b) the external column removed after seismic strengthening.



(a) (b)

Fig. 18. Displacement time-history of FRAME2 with (a) the external column removed and (b) the external column removed after seismic strengthening.

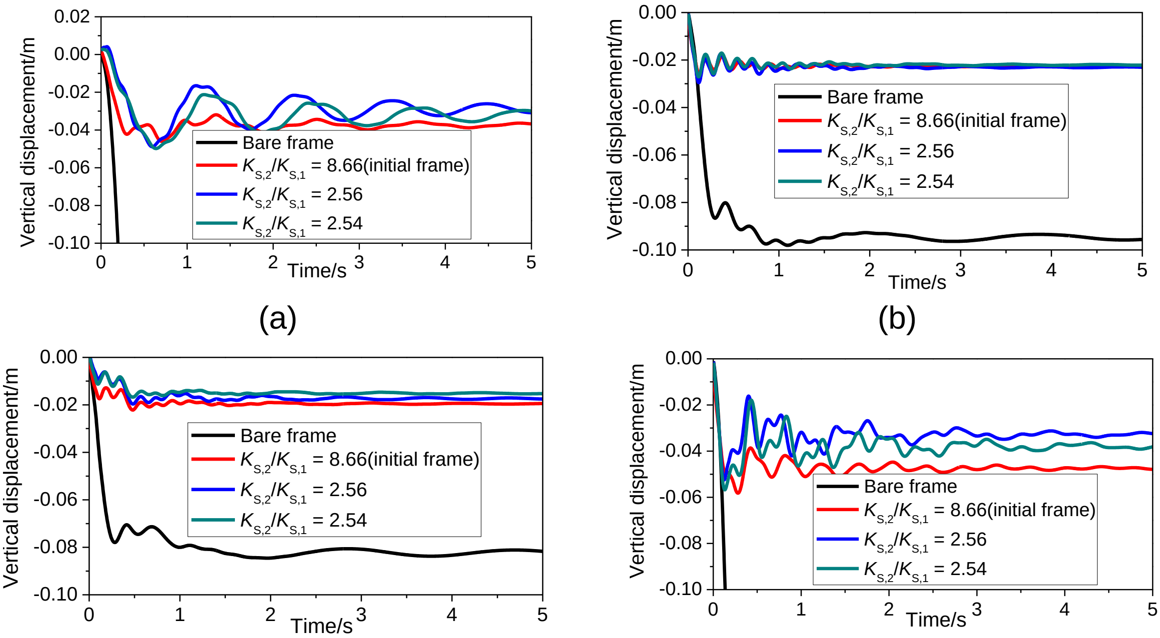
In this section, the analysis is implemented in two steps. Firstly, an pushover procedure is applied on the structure to consider the rare seismic action intensity, and then the structural progressive collapse resistant capacity is evaluated. Due to the residual deformations are usually exist after earthquake excitation and the single direction pushover analysis procedure, four cases with A, B, C and D columns removed (See Fig. 8) are all analyzed. Fig. 19(a)-(d) and Fig. 20(a)-(d) show the vertical displacement time-history results of FRAME1 and FRAME2 for each column failure case, respectively. These figures illustrate that the structural progressive collapse resistant capacities are not significantly decreased after subjected to the rare earthquake excitations, where only larger displacements are observed compared with the initial structures.



(a) (b)

(c) (d)

Fig. 19. Displacement time-history of FRAME1 with the earthquake-induced damage (a) Column A removed, (b) Column B removed, (c) Column C removed, (d) Column D removed.



(a) (b)

(c) (d)

Fig. 20. Displacement time-history of FRAME2 with the earthquake-induced damage (a) Column A removed, (b) Column B removed, (c) Column C removed, (d) Column D removed.

5. CONCLUSION AND DISCUSSION

The seismic and progressive collapse potentials of RC infilled frames with a soft story are studied in this paper. Two typical infilled frames designed to two different seismic intensity levels are analyzed to evaluate their seismic and progressive collapse performance. On this basis, a strengthening strategy by modifying the lateral stiffness ratio of second story to first story is suggested and its effectiveness on both two collapse potentials is examined. The main conclusions can be summarized as follows:

(1) For a soft-story fame, improper design of infill walls may lead to most of the plasticity developments occurred in the columns rather than in the beams. However, infill walls seem to be beneficial to the progressive collapse capacity, although they will decrease the ductility.

(2) The seismic strengthening by control the lateral stiffness ratio of second story to first story proves to be effective to its seismic collapse capacity. The seismic strengthening of the infilled frame can also improve its progressive collapse capacity by further increasing the ultimate resistant load, and can also increase the ductility capacity. A reasonable lateral stiffness ratio of second story to first story can get an improvement on both the seismic performance and the progressive collapse resistant capacity. In practical design of the infilled frames, the design with a proper lateral stiffness ratio of second story to first story is preferred.

(3) The earthquake-induced structural damage will increase the risk of the following possibly occurred progressive collapse. However, this effect may be not obvious if the earthquake-induced structural damage is not severe enough. For the two code-design frames in the study, after suffered to rare seismic action, the vertical displacements in the progressive collapse process are only a little larger than that of the initial frames, and collapse mechanism is not changed. In the process of the design, if the idea of this strengthening can be properly considered, a soft-story frame will provide the sufficient capacity on both seismic collapse and progressive collapse.

ACKNOWLEDGEMENTS

This research project is supported by the National Natural Science Foundation of China (Grant No. 51008101, 51278150), and the China Postdoctoral Science Foundation (No. 2012T50361). The financial supports from the research funds are greatly appreciated by the authors.

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