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Effect of brick infill panel on the seismic safety of reinforced concrete frames under progressive collapse

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Abstract. Structural safety has always been a key preoccupation for engineers responsible for the design of civil engineering projects. One of the mechanisms of structural failure, which has gathered increasing attention over the past few decades, is referred to as ʹprogressive collapse' which happens when one or several structural members suddenly fail, whatever the cause (accident, attack, seismic loading(.Any weakness in design or construction of structural elements can induce the progressive collapse in structures, during seismic loading. Masonry infill panels have significant influence on structure response against the lateral load. Therefore in this paper, seismic performance and shear strength of R.C frames with brick infill panel under various lateral loading patterns are investigated. This evaluation is performed by nonlinear static analysis. The results provided important information for additional design guidance on seismic safety of RC frames with brick infill panel under progressive collapse.

Keywords: progressive collapse; brick infill panel; Lateral loading; seismic performance; shear strength

1. Introduction

1.1 Background

Progressive collapse first attracted the attention of engineers from the structural failure of a 22-story apartment building at Ronan-Point, London, UK, in 1968. The terminology of progressive collapse is defined as "the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it". After the event of 11 September 2001, more and more researchers have started to refocus on the causes of progressive collapse in building structures, seeking ultimately the establishment of rational methods for the assessment and enhancement of structural robustness under extreme accidental events. In the United States the Department of Defense (2005) and the General Services Administration (2003), provide detailed information and guidelines regarding methodologies to resist progressive collapse of building structures. Both employ the alternate path method (APM) to ensure that structural systems have adequate resistance to progressive collapse. Izzuddin *et al.*

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(2008) in two papers presented a design-oriented methodology for progressive collapse assessment of multistory buildings. The proposed assessment framework consists of three stages: nonlinear static response of the damaged structure under gravity loading, a simplified dynamic assessment to establish pseudo static curves, and ductility assessment of the connections. The second paper details the application of the new approach to progressive collapse assessment of real steel-framed composite multistory buildings. Kim *et al*. (2009) studied the progressive collapse-resisting capacity of steel moment resisting frames using alternate path methods recommended in the GSA and DoD guidelines. The linear static and non-linear dynamic analysis procedures were carried out for comparison. It was observed that the nonlinear dynamic analysis provided larger structural responses and the results varied more significantly. However the linear procedure provided a more conservative decision for progressive collapse potential of model structures. Khandelwal *et al*.(2009) studied the progressive collapse resistance of seismically designed steel braced frames with validated two dimensional models. Two types of braced systems are considered: namely, special concentrically braced frames and eccentrically braced frames. The study is conducted on previously designed 10-story prototype buildings by applying the alternate path method. The simulation results show that while both systems benefit from placement of the seismically designed frames on the perimeter of the building, the eccentrically braced frame is less vulnerable to progressive collapse than the special concentrically braced frame. Because of high complexity of masonry infill panels and some efficient factors, few investigations have been performed about this issue. For the first time, Sasani (2008) has conducted field test to investigate the dynamic response of a RC building with brick-infill panels subjected to sudden column loss. The brick wall was modeled by shell or equivalent compression-strut elements and the simulation results were compared. For conventional RC buildings, the brick-infill panels are usually adopted for interior partitions. Another study of infill panel was carried out by Tsai *et al*. (2009). In this study, the brick-infill panels are simulated by compression struts to clarify their effect on the progressive collapse potential of an earthquake-resistant RC building. Linear static analyses are conducted to investigate the variation of demand-to-capacity ratio (DCR) of beam-end moment and the axial force variation of the beams adjacent to the removed column. Study results show that contribution of the brick infill to DCR reduction depends on its location and dimensions. More significant reduction is achieved as the brick-infill panels are filled in the structural bay adjacent to the removed column. In this paper, seismic performance and shear strength of R.C frames with infill panel under various lateral loading patterns are investigated. This evaluation is performed with nonlinear static analysis. The results provided important information for additional design guidance on seismic safety of RC frames under progressive collapse.

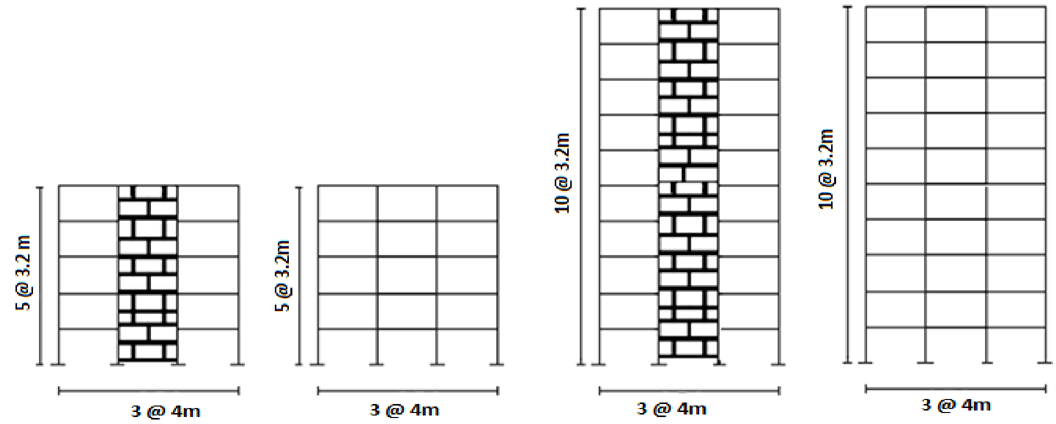
1.2 Objectives and scope

Investigation the effect of infill panels on progressive collapse has recently attracted notable attention. Masonry infill panels are widely applied in structures due to architectural and structural purposes. Since, the infill panels have significant influence on structure response against lateral loads, this question is encountered that, whether infill panels have any effect on structure response under the progressive collapse. A progressive collapse involves a series of local failures that lead to partial or total collapse of a structure. Any weakness in design or construction of structural elements can cause the local failure that lead to progressive collapse in structures, during lateral loading particularly seismic loading. Most researches include the assessment of the potential progressive collapse under gravity load (Izzuddin *et al*. 2008, Khandelwal *et al*. 2009, Kim *et al*. 2009), and others focus on deterministic approaches to obtain the most accurate method (GSA 2003, UK 2004, DOD 2005). Some researches include assessment the effect of brick-infill panel on the progressive collapse potential under gravity loading (Sasani 2008; Tsai *et al*. 2009), and no attention has been given to progressive collapse under lateral loads. Thus in this paper the effect of infill panels on potential progressive collapse in RC frames under different lateral load patterns has been investigated. Experiences from various earthquakes show that infill walls have important effect on structure seismic response. In addition, performance based design reliably represents the response of a structure under lateral loading. Hence in this study the influence of infill panels in seismic performance of RC frames under progressive collapse, based on performance level is investigated. After the analyses are carried out seismic response of the frames including shear strength index (R), generation of plastic hinges, story drift and performance level is obtained. Then the results are compared with the required criteria in FEMA356 and GSA and seismic behavior of the frames is evaluated. The results indicate that existence of infill panel prevent the progression of failure and restrict it within a localized area.

2. Analytical modeling of model structure

2.1 Modeling of the RC frames

The RC models are 5 and 10-story moment-resisting frames with low ductility. There are three bays with 4m span and the story height is 3.2m (Fig. 1). The frames were designed for a dead load (DL) of 20 KN/m and a live load (LL) of 3KN/m for the roof and 4KN/m for other floors (ACI. 2008).All the beams and columns are designed and detailed according to seismic code requirements. Tables 1 and 2present the section dimensions of the RC members for the building and Table3 presents material properties for concrete and reinforcements used in the models. A beam-column frame model is constructed for the RC building using the SAP2000 commercial program (CSI, SAP 2000. 2006). It is assumed that the model is fixed on the ground.



(a) 5-story frame (b) 5-story frame (c) 10-story frame (d) 10-story frame

Fig. 1 Modeling of RC frames

Table 1 Dimensions of RC member sections in 5-story frame

Floor	Column (cm)	Beam (cm)
5F	30 × 30	30 × 30
4F	35 × 35	35 × 35
3F	35 × 35	40 × 40
1~2F	40 × 40	40 × 40

Table 2 Dimensions of RC member sections in 10-story frame

Floor	Column (cm)	Beam (cm)
10F	35 × 35	30 × 30
9F	40 × 40	35 × 35
8F	45 × 45	40 × 40
7F	45 × 45	40 × 40
6F	50 × 50	50 × 50
5F	50 × 50	50 × 50
4F	55 × 55	50 × 50
3F	55 × 55	50 × 50
2F	55 × 55	50 × 50
1F	60 × 60	55 × 55

Table 3 Input data for concrete and reinforcement

Property					
Density	Modulus of elasticity	Poisson ratio	Compressive strength (yield stress)	Bending yield stress (for the reinforcements)	Shear yield stress (for the stirrups)
ρ	E	ν	f_c	f_y	f_{ys}
(ton/m^3)	(GPa)	(MPa)	(MPa)	(MPa)	(MPa)
2.4	21.8	0.2	21	400	300

2.2 Modeling of the brick-Infill

The brick-infill wall was modeled by equivalent compression-strut elements. They are often considered as non-structural elements and only their weight is accounted for in structural design. However, from several experimental studies on brick-infill RC frames, it was observed that the brick wall may contribute to the horizontal seismic resistance of RC frame. Hence, it may help to reduce the progressive collapse potential for RC building (Tsai *et al*. 2009). When a frame equipped with infill panel with no connection or anchorage between the frame and infill panel, infill panel may be separated from the surrounding frame under the lateral loading (Fig. 2a). In this condition, only the corners under the lateral loading are attached to the frame (Fig. 2b). Hence, the brick infill wall is simulated by the compression-strut model suggested by the FEMA 356 (Fig. 3). The equivalent width of the compression strut, a, is modified as:

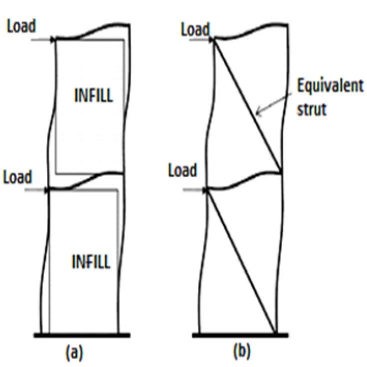


Fig. 2 Compression-strut under horizontal loading

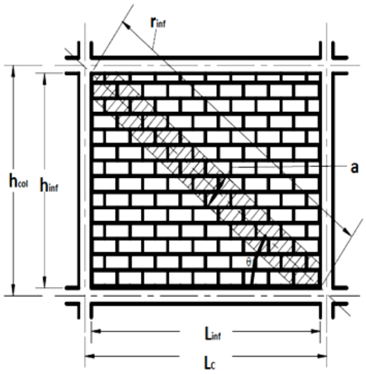


Fig. 3 Panel of the brick infill wall

Table 4 Dimensions of compression-strut in 5-story frame

Floor	Width of the compression- strut	Thickness of infill panel	Diagonal length of infill panel
	a (cm)	t_{inf} (cm)	r_{inf} (cm)
5F	42.7	30	476
4F	45.55	30	476
1~3F	47.9	30	476

$$a = 0.175\,(\lambda_1 h_{col})^{-0.4} r_{inf} \tag{1}$$

where

Where h_{col} and $r_{inf}$are column height between centerlines of beams and the diagonal length of infill panel, respectively t_{inf} is the thickness of infill panel and strut. h_{inf} and E_{me} are respectively the height of infill panel and expected elastic modulus of infill panel. E_{fe} is the expected elastic modulus of frame material. I_{col} is the moment of inertia of column. θ is the angle whose tangent is the infill length-to-height aspect ratio, in radian. As recommended by FEMA356, E_{me} is calculated as 550 f_m, where f_m is the compressive strength of the infill (and assumed as 4000 KPa in this study), Therefore, the elastic modulus of the infill panel E_{me} is 2200 MPa. Table 4 presents the dimensions of compression strut of 5-story frame.

3. Analysis procedure and loading

3.1 Modeling of the brick-Infill

The non-linear static analysis (Pushover Analysis) gives better understanding and more accurate seismic evaluation of buildings as the progression of damage and failure can be traced.

The step by step procedure of the pushover analysis shows the performance level of the building components and also maximum base shear carrying capacity of the structure. It is an efficient method for the performance evaluation of a structure subjected to seismic loads. Using these procedures this report is detailed with modeling aspects of the hinge behavior, acceptance criteria and assessment the performance level (Zine *et al*. 2007, Jianmeng *et al*. 2008, Kadid *et al*. 2008).

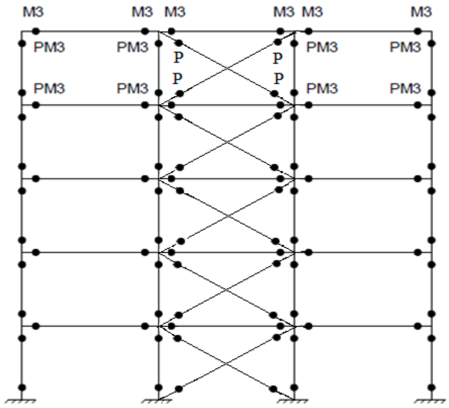


Fig. 4 Location of plastic hinges

3.2 Nonlinear hinge properties

In order to conduct nonlinear analyses on the structure and modeling its nonlinear behavior, plastic hinges must be assigned to the members according to their behavior and then nonlinear analyses are run. Internal stresses in beams, columns and infill panels of the moment resisting frames are flexural, axial-flexural and axial types respectively. Plastic deformations in these members, under earthquake loading appear as plastic hinges in their end points. Thus flexural plastic hinges (M) are assigned to the end points of the beams while axial-flexural plastic hinges (PMM) are assigned to the end of columns and axial hinges (P) are assigned to two end points of the diagonal element of the infill panel (Fig. 4). Plastic hinges for all structural elements are defined at 0.05 L in end points (where L is the element length). Table 5 presents the nonlinear hinge properties.

3.3 Nonlinear hinge properties

United States Government Standards Federal buildings in the USA are generally designed in line with the GSA guidelines (GSA 2003). These guidelines were developed to provide minimum requirements for mitigating the risk of progressive collapse particularly for new and existing facilities of ten stories or less. They employ the alternate load path method and despite the dynamic nature of instantaneous member removal, they promote an equivalent nonlinear static analysis technique.

According to this approach, the following load combination is proposed:

$$2(D_L+0.25L_L) \tag{3}$$

Where, D_L is Dead load and L_L is live load and the amplification factor of 2 is used to approximate the dynamic effects associated with sudden member loss. In nonlinear static analysis a series of step-by-step procedure was followed to model the structures. The first step is to apply the load combination (D_L+0.25L_L) and a specific lateral load pattern to the structure. Then a vertical load bearing member is removed completely and to account for the dynamic effect of this removal, an additional load combination of (D_L+0.25L_L) is applied to spans adjacent to the

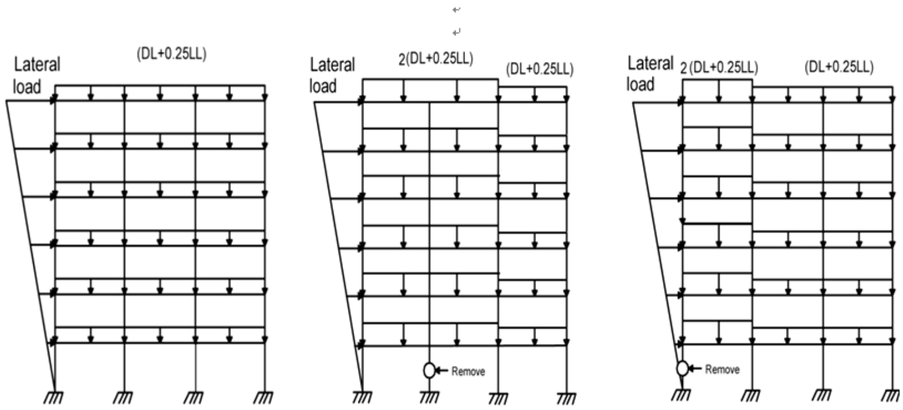


Fig. 5 Applied load for analysis of progressive collapse in static procedure (GSA 2003)

removed member. In the next step the applied load on the structure is increased incrementally until the maximum load or the target displacement is attained. It should be noticed that removing column completely and suddenly has more critical effect on progressive collapse potential of model structures. Finally, if the performance level of the structure is restricted to the allowable limit specified in the FEMA 356, the structure is deemed safe with respect to progressive collapse and the design is considered adequate, otherwise the structure needs to be redesigned. All samples are evaluated in three states: (a) Without local failure; (b) Removal of the second column; (c) Removal of the corner column (Fig. 5).

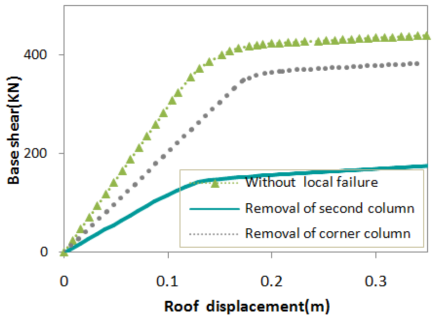
4. Results and discussions

4.1 Capacity curves of reinforced concrete frames

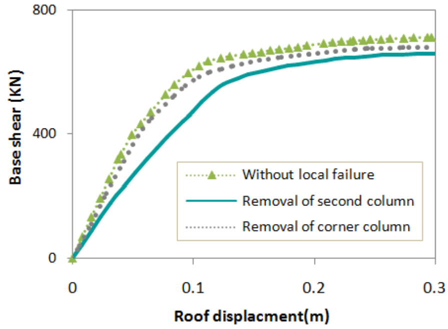
In pushover analysis, lateral loading is applied to the structure gradually and this loading increases according to a specific loading pattern until displacement of the control point reaches to limit target displacement. Rehabilitation Instruction states that in order to perform nonlinear static analysis and take to account of the most critical condition, at least two distribution of lateral load should be applied. So in this study the RC frames have been analyzed under three separate lateral

(a) Triangular pattern (b) Uniform pattern (c) Mode1 pattern.

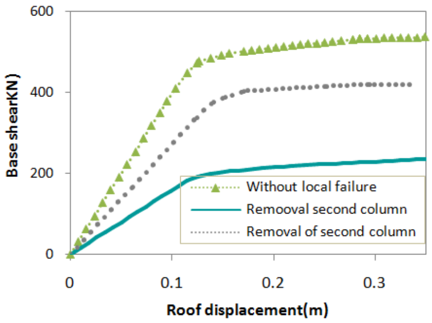
Fig. 6 Applied lateral loading distribution patterns



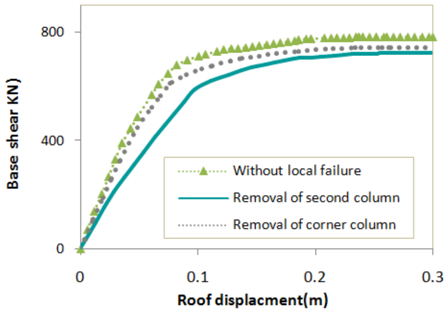
(a) 5-story frame under triangular pattern



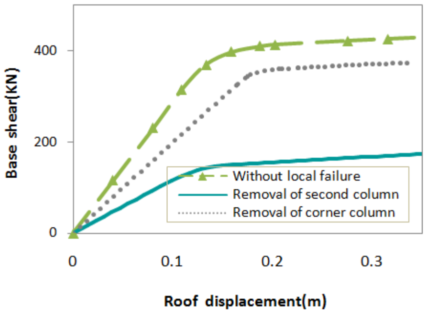
(b) 5-story frame with brick infill panelunder triangular pattern



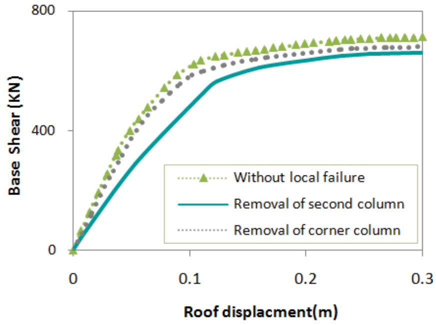
(c) 5-story frame under uniform pattern



(d) 5-story frame with brick infill panel under uniform pattern



(e) 5-story frame under mode1 pattern



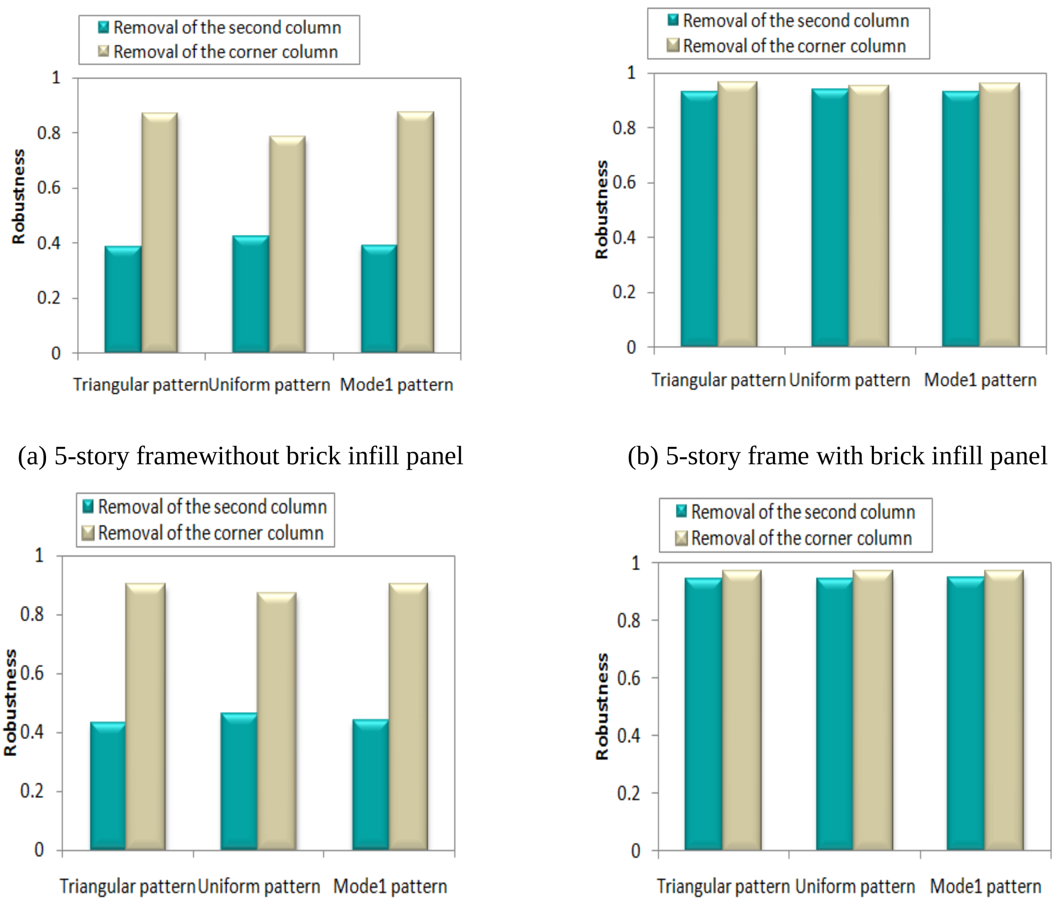
(f) 5-story frame with brick infill panelunder mode1pattern

Fig. 7 Capacity curves of reinforced concrete frames

load pattern namely triangular and uniform pattern (as recommended in FEMA), and a lateral load distribution proportional to the first vibration mode in the direction under study (as recommended in ASCE), (Fig.6). Experiences from various earthquakes show that infill walls have important effect on structure seismic response. Hence in this study the influence of infill panels in seismic performance of RC frames under progressive collapse is investigated. Capacity curves show that these walls enable the frames to bear larger lateral loads. In the presence of the infill panel, local failure caused by removal of either corner or second column results in only a negligible reduction in the base shear so structure is able to retain its shear strength. Also base shear at the target displacement derived from the uniform lateral loading distribution is larger than two others types of loading, (Fig. 7).

4.2 Quantitative assessment of structural robustness

Progressive collapse of a structure refers to the condition when the failure of a local component leads to global system failure. Structural robustness is defined as the ability of resisting progressive collapse, and indicates the overall performance of the damaged structure assuming a load-bearing element removed. The current progressive collapse analysis procedures can only give a qualitative assessment of the robustness of the overall structural system. However, to conduct a



(a) 5-story framewithout brick infill panel

(b) 5-story frame with brick infill panel

(c) 10-story framewithout brick infill panel

(d) 10-story frames with brick infill panel

Fig. 8 The robustness indices of the damaged structure under various lateral loading

Table 8 Performance level of building

Performance level			
Operational Level (A-1)	Immediate occupancy level (B-1)	Life safety level (C-3)	Collapse prevention level (E-5)
O	IO	LS	CP

Table 9 Structural performance levels and drift (Base on FEMA-356, Table C1-3)

Elements	Collapse prevention S-5	Life safety S-3	Immediate occupancy S-1
Concrete moment frames	4% Transient or permanent	2% Transient	1% Transient
Steel moment frames	5% Transient or permanent	2.5% Transient	0.7% Transient
Braced steel frames	2% Transient or permanent	1.5% Transient	0.5% Transient
Concrete walls	2% Transient or permanent	1% Transient	0.5% Transient

progressive collapse control design, we need to know the quantitative results of structural robustness and the reserve load-bearing capacity of the damage structure (Baker *et al*. 2008, Lu *et al*. 2010). In this paper, the quantitative value of the robustness of a structure is obtained by index R:

$$R= \frac{V\ \text{damaged}}{V\ \text{design}} \tag{4}$$

where R is the residual reserve strength ratio, $V_{damaged}$ is the ultimate shear force of the damaged structure at target displacement, V_{design} is shear force of the structure without local damage at target displacement.

4.2.1 Assessment of structural robustness against the progressive collapse

Values of R index under various lateral loading patterns of 5-story reinforced moment frame structures, with and without infill panel are presented in Tables 6 and 7 respectively. For all lateral load patterns robustness roughly follow the same trend. In addition in all cases removal of the second column leads to a larger reduction of base shear. This reduction is particularly significant when the failure happens in absence of the infill. For example it can be observed from Table 6 that the removal of the second column under triangular pattern decreases the value of base shear from 431 to 165 indicating a reduction 62%, in other words the damaged structure is able to retain 38% of its initial shear strength. While frames equipped with brick infill experienced a 7% and 4% base shear reduction, respectively for removal of second and corner column (Table 7), it shows that local failure in the presence of infill reduces the base shear very slightly. Infill panels favorably affect the lateral load-bearing capacity of structures and keep the structure shear strength close to its value prior to the failure. As well as increasing the numbers of stories index of R is increased (Fig. 8).

4.3 Seismic evaluation of structures

4.3.1 Performance level

Building performance can be described qualitatively in terms of the safety afforded building occupants during and after the event; the cost and feasibility of restoring the building to pre-earthquake condition. These performance characteristics are directly related to the extent of damage that would be sustained by the building (FEMA 356). The extent of damage to a building is categorized as a Building Performance Level. These performance levels are tabulated inTable 8.

4.3.2 Drift

The concept of drift is used to control the lateral displacement. In this section, story drift of each structure at target displacement is calculated and compared with the allowable limits specified in FEMA356. These values are only used to qualitatively assess of structures behavior at desired performance level. Performance level is the condition of structure after earthquake which presents the extent of damage imposed by earthquake. Performance levels are tabulated in Table 8 and control limitations of drift of various types of structures are tabulated in Table 9. Transient displacement (Δ_1) is maximum predicted lateral displacement of stories which happens during design earthquake. Permanent displacement (Δ_2)is maximum lateral displacement which is caused by an earthquake and remains due to plastic behavior of materials in the structure. These concepts are shown in Fig. 9.

Where h is the height of story and Δ_n is the lateral displacement of nth story at target displacement. According to Fig. 10, Table 10 and 11, story drifts in RC frame without brick infill panel under different lateral load pattern are limited the value represented LS performance level (0.02), and the story drift values along the height, particularly for the 10-story frame are close together. When infill are added to the midspan of 5-story frame, story drift in upper stories are significantly reduced even to as low as 0.01, which is the story drift value attributed to IO performance level. On the other hand, story drifts in lower story exceed the intended limit (LS), approaching even to the value representing CP performance level. So infill panels improve performance structures in upper stories and localize the imposed damage in lower stories. This effect is more sever when uniform lateral load is applied. It is obvious that increase of story level improve the performance of the structure, especially in lower stories.

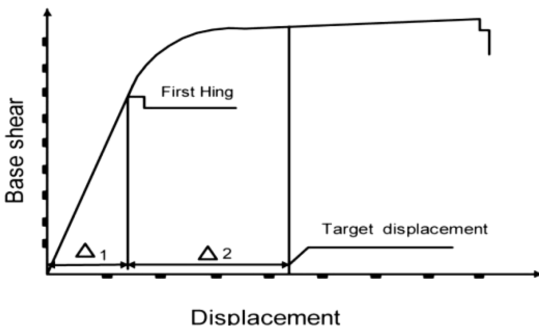
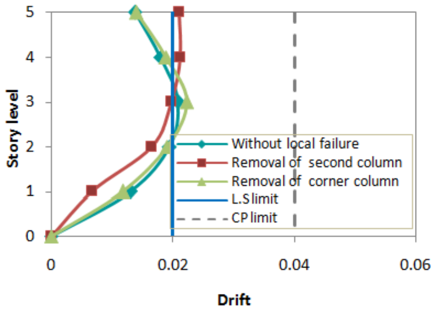
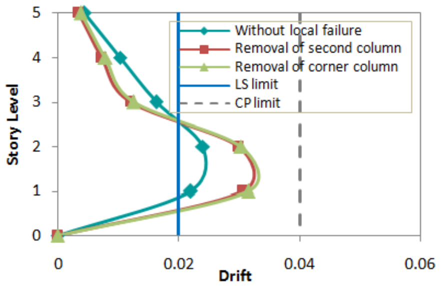


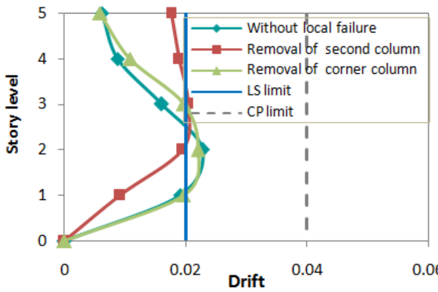
Fig. 9 Capacity curve



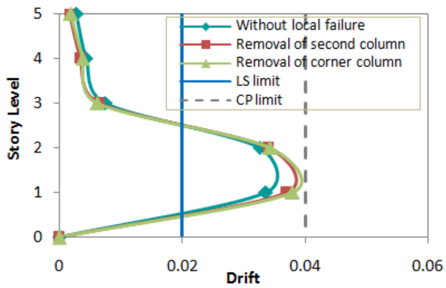
(a) 5-story structure under triangular pattern



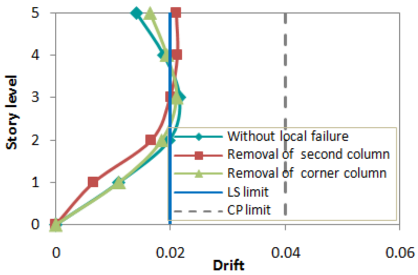
(b) 5-story structure with brick infill panelunder triangular pattern



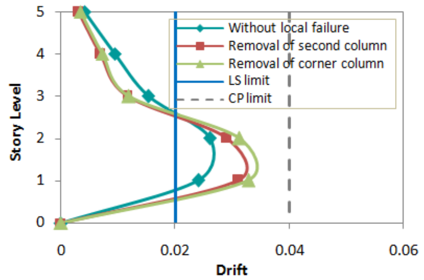
(c) 5-story structure under uniform pattern



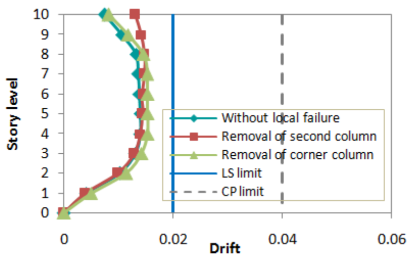
(d) 5-story structure with brick infill panel under uniform pattern



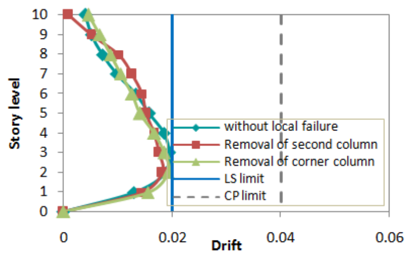
(e) 5-story structure under mode1pattern



(f) 5-story structure with brick infill panel under mode1 pattern



(g) 10-story structure under triangular pattern



(h)10-story structure with brick infill panel under triangular pattern

Fig. 10 Story drift at target displacement

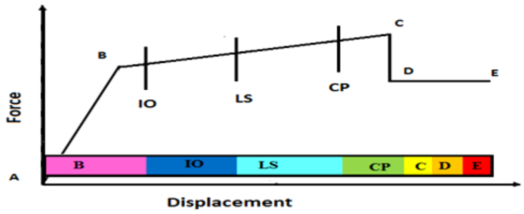
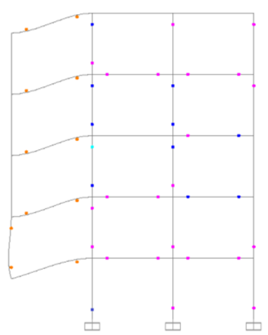
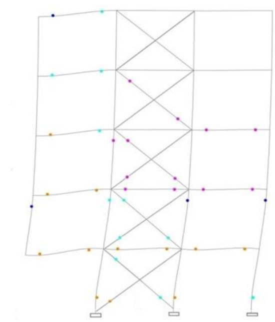


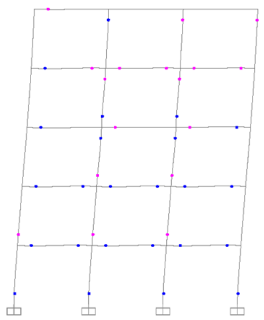
Fig. 11 Force-displacement curve of hinges with colored codes



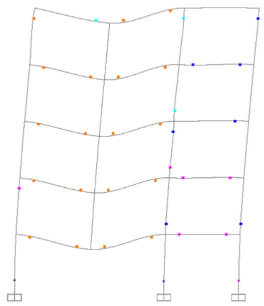
(a) 5-story frame without local failure



(b) 5-story frame with removing second column



(c) 5-story frame with removing corner column



(d) 5story frame with brick Infill panel removing corner column)

Fig. 12 Plastic hinges status at target displacement under triangular pattern

Table 10 Drift of story in 5-story frame under triangular pattern

	Drift				
Story Level	Without local failure	Without local failure	Removal of corner column	LS limit	CP limit
5	0.0137	0.0211	0.014	0.02	0.04
4	0.0179	0.0212	0.0189	0.02	0.04
3	0.0207	0.0197	0.0222	0.02	0.04
2	0.0194	0.0165	0.0190	0.02	0.04
1	0.0130	0.0067	0.0117	0.02	0.04

Table 11 Drift of story in 5-story frame with brick infill panel under triangular pattern

	Drift				
Story level	Without local failure	Without local failure	Removal of corner column	LS limit	CP limit
5	0.004	0.003	0.0037	0.02	0.04
4	0.010	0.007	0.0078	0.02	0.04
3	0.016	0.012	0.0125	0.02	0.04
2	0.028	0.030	0.0303	0.02	0.04
1	0.026	0.032	0.0315	0.02	0.04

4.3.3 Evaluation of hinges in performance levels

In SAP2000, non-linear behavior is assumed to occur within frame elements at concentrated plastic hinges. The pushover analysis consists of the application of gravity loads and a representative lateral load pattern. The lateral loads are applied monotonically in a step by step non-linearstaticanalysis.The applied lateral loads are accelerations in the direction under study representing the forces that would be experienced by the structures when subjected to ground shaking. Under incrementally increasing loads, some elements may yield sequentially. Consequently, at each event, the structure experiences a stiffness change as shown in Fig.11, where IO, LS and CP stand for immediate occupancy, life safety and collapse prevention respectively (Vijayakumar *et al*. 2011). In each stage of nonlinear static analysis, hinge status indicating the performance level of the structure identified by colored codes. Location of plastic hinges is shown in Fig. 12. It can be clearly seen that by removal of the corner and second column in absence of infill panel, plastic hinges occur in almost all stories. Also beams of the span adjacent to the removed column in all stories exceed the performance level of CP. However more plastic hinges are formed in columns when a corner column is removed which is more critical than the removal of a second column in which most columns do not yield and will not enter the plastic region. Fig. 12d, shows the formation of plastic hinges in a frame equipped with infill panel in the midspan when a corner column is removed. As depicted in Fig. 12d, in lower stories, plastic hinges occur in the diagonal struts and RC members while no plastic hinges are formed in upper stories, (story 4, 5). In other words, failure of RC members is localized in lower stories and restricts the spread of local damage.

5. Conclusions

In this study, effect of local failure on the seismic safety of RC frames with infill panel under lateral loading was evaluated. The nonlinear elastic analysis presented in GSA guidelines was the basis of this study. According to the results obtained from this study, the following conclusions could provide important information for additional design guidance on effect of infill panel on seismic safety of RC frames under progressive collapse:

- Failure of the second column in the frame without infill panel significantly decreases the strength of the structure to transfer shear of damaged component, but addition of the infill panel will compensate for this defect, largely increasing the structure's shear capacity.

- It was observed that the progressive collapse potential decreased as the number of stories increased, because more structural elements exist to carry and transfer the load of failed element.
- Evaluation of capacity curves and R index of the studied frames show that addition of infill panel increase the structure shear strength and improve performance of structures in upper stories with preventing the spread of failure and localize the imposed damage in lower stories.
- Structures with infill panel designed based on current seismic guidelines are not in appropriate safety margin against progressive failure in lower stories. Hence in these structures should be performed a second analysis by eliminating the critical column.
- Taking everything into account, infill panels profoundly affect the overall seismic response and performance of buildings against progressive collapse under lateral loading. Therefore for building with high importance that are prone to progressive collapse, the effect of infill panels should not be overlooked when a rehabilitation design is provided and the performance level is evaluated.

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