# Seismic Response of Infilled RC Frames Designed Considering Local Wall-Frame Interactions

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## Abstract

It is well known that the interactions between the frames and the infill walls can have significant effects on the seismic response especially for low-rise frames. On one hand, the infill walls can increase the strength and stiffness of the frame, which result in higher seismic resistance. On the other hand, the local interaction between the infill wall and the frame due to the compressive diagonal strut in the infill wall can lead to increased shear demand in the column. This can lead to an undesirable failure mode such as shear failure of the column leading to loss of frame strength at low drift levels. One appealing approach for considering infill walls is by preventing all the undesirable failure modes induced by the wall-frame interaction while not considering the advantages from the increase in strength and stiffness due to the infill wall. Based on this approach, infilled frames are analyzed and designed as if they were bare frames as commonly done in design practice. Columns are then checked for increased shear demand at the local level to counteract the adverse effects of the infill walls. In this study, one such design strategy is investigated. The design approach is first presented including a method to estimate the increased shear demand in the column due to local wall-frame interaction. The approach was used to design RC frames with infill walls located in Thailand. The performance of these frames was assessed using nonlinear static and dynamic analyses. The performance of the frames and the failure modes were discussed in comparison to those of the frames designed without considering the local interactions. The merit of this approach as well as its limitations were discussed and summarized.

*Keywords: RC frames, infill wall, seismic design method, shear failure, wall-frame interaction*

# 1. Introduction

RC frames with masonry infill walls are widely used in many seismically active areas especially for low- to medium-rise buildings. For design purposes, the complex interaction between the frame and infill wall is treated differently in various national building codes [1] and clear design guidelines are still lacking. In general, the code approaches for considering the infill walls fall into two categories: those that consider the infill wall-frame interaction and those that require the infill walls to be isolated from the main frames, thereby eliminating this interaction. It is well known that infill walls can increase the strength and stiffness of the frame, which result in higher seismic resistance. On the other hand, the local interaction between the infill wall and the frame often can lead to increased demand in the surrounding members. This can lead to an undesirable failure mode such as shear failure of the column leading to loss of load carrying capacity at low drift levels. For this reason, if one can harness the strength and stiffness of the infill wall while preventing the failure of the surrounding members, the frame can gain significant seismic resistance.

In this study, an effective design strategy to consider infill-frame interaction is investigated. The approach is based on plastic mechanism analysis of the column considering the infill wall-frame interaction recently proposed by Wararuksajja et al. (2020) [2]. In this approach, the infilled frames are analyzed and designed as if they were bare frames as commonly done in design practice. Additional steps are then applied to prevent local failure of the surrounding columns. The columns are checked for increased shear demand at the local level at different response levels, and necessary design measures are applied to counteract the adverse effects of the infill walls. The merit of this approach is that it can be readily applied without radically changing common design practices. However, the adverse effects due to wall-frame interaction are carefully considered in a systematic manner.

In this paper, the main concept for the design approach is briefly reviewed. The approach was applied to design prototype RC frames with infill walls located in Thailand. The performance of these frames was assessed using nonlinear static and dynamic analyses. The performance of the frames and the failure modes were discussed in comparison to those of the frames designed without considering the infill wall and without considering the local interactions. Based on the performance evaluation, the effectiveness of the proposed design approach is discussed.

# 2. Design Approach to Consider Local Infill-Frame Interaction

Recently, Wararuksajja et al. (2020) [2] proposed a design method to prevent column shear failure due to wall-frame interactions without considering the global effect of infill walls. The strength of the column is first still designed following code-based frame analysis. Then, the column shear demand and column shear capacity due to wall-frame interaction are examined at different damage states of the frame response. In this method, the infill wall is represented by an equivalent compressive strut. The strut axial force capacity ($C_i$) can be estimated by a chosen method depending on the expected failure modes of the wall [3]-[12]. Once the internal force of the wall reaches the strut capacity, the damage will commence. Fig. 1 illustrates the response of the infilled RC frame in different states. In the initial state, before wall cracking or crushing, a single equivalent diagonal strut is formed between the loaded corners, as shown in Fig. 1a. This state is usually associated with the response prior to reaching the peak lateral resistance of the system due to the combined frame and the wall resistance. Beyond this state, when the force in the equivalent strut reaches strut's capacity, corner crushing occurs normally near the upper-end regions of the column (Fig. 1b). After corner crushing, the response is bested represented by two adjacent struts with one strut acting below the diagonal line on the front column below the damaged area and the other strut acting towards the beam above the diagonal line. The magnitude of force in the struts in this state is smaller the original strut. The force of offset struts is assumed to be $\alpha C_i$ where α is a reduction factor to decrease from original one. After this point, the failure modes depend on the relative of the column and the remaining strut force. For a relatively strong frame, the opening gap will propagate downward along the column height and the strut force acting on the column continuously shifts downward, corresponding to increasing drift (Fig. 1c). However, for a relatively weak frame, shear failure of the column could be expected due to the bracing of the remaining wall, as shown in Fig. 1d.

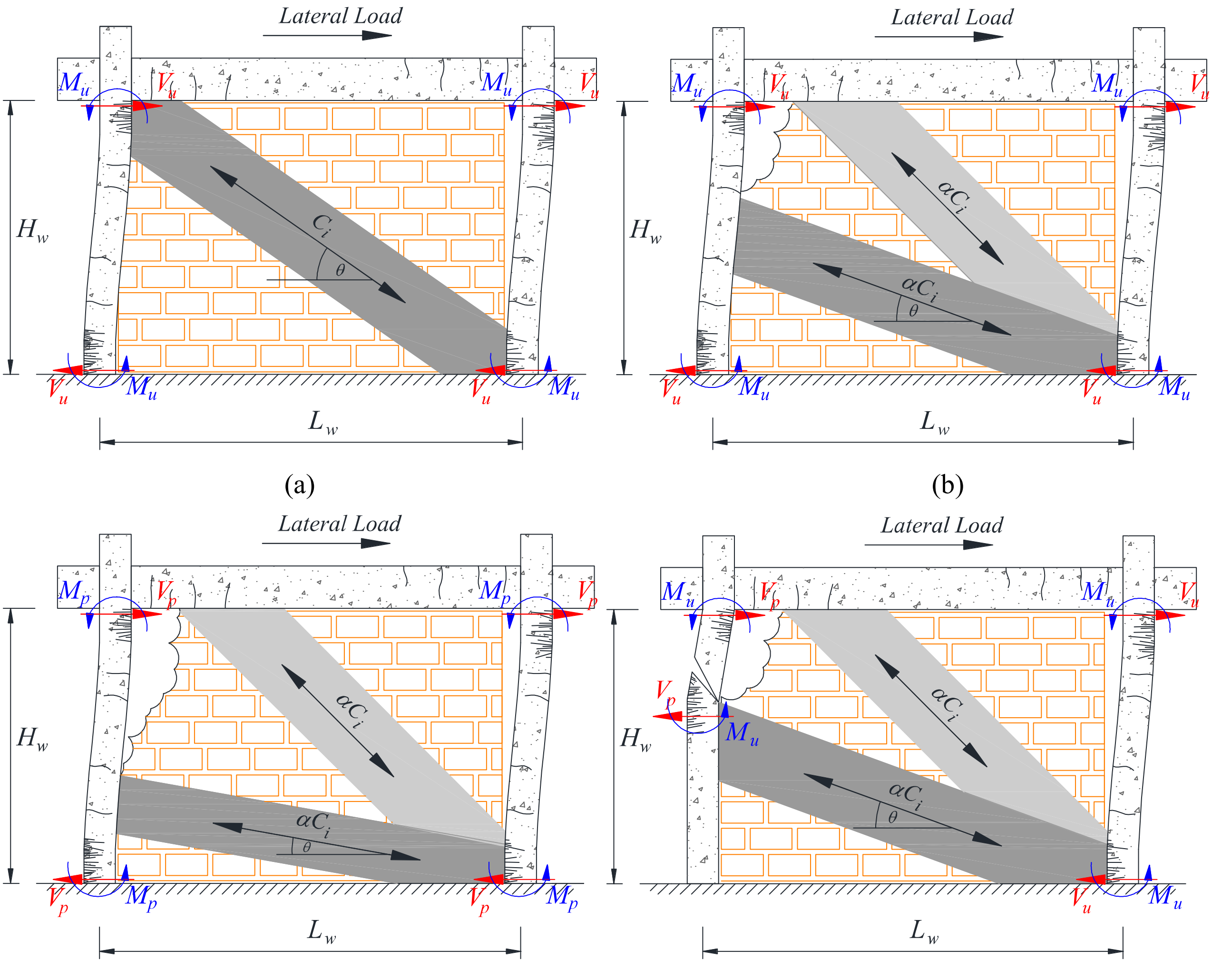



Fig. 1 – Frame-infill behavior: (a) initial state; (b) corner crushing; (c) relatively strong frame mechanism; and (d) relatively weak frame mechanism [2]

Wararuksajja et al. (2020) [2] proposed a design method based on equilibrium of an isolated column. The column shear demand and column shear capacity are examined at different damage states of the infill wall. Two possible mechanisms of the column are considered including column-infill (for a frame with relatively strong columns) and column mechanisms (for a frame with relatively weak columns) as depicted in Fig. 2. For the column-infill mechanism, the crushing zone of infill wall continuous propagate until the plastic hinges forms in the column ends (Fig. 2a). The column shear ($V_a$) can be estimated based on the plastic hinges at the ends of the columns and the compressive force from the residual strut as shown in Eq

$$V_a = \frac{2M_p}{H_w} + \frac{\alpha C_i \cos\theta (H_w - a)}{H_w} \quad (1).$$

$$V_a = \frac{2M_p}{H_w} + \frac{\alpha C_i \cos\theta (H_w - a)}{H_w} \tag{1}$$



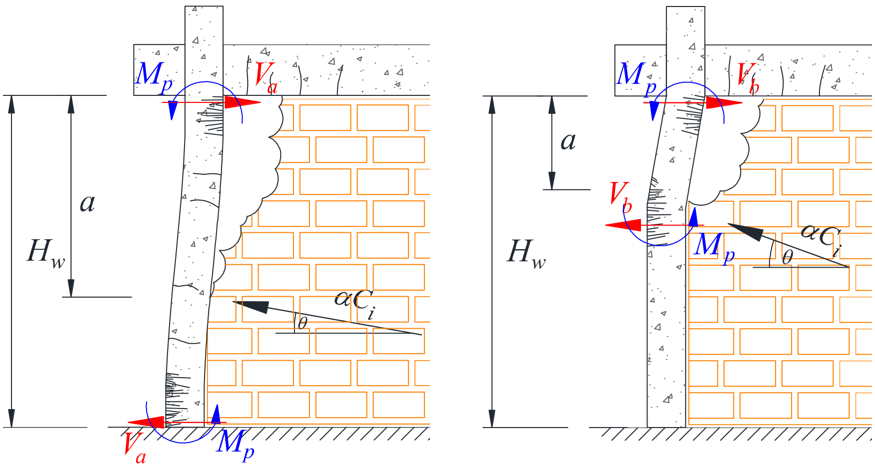
where $C_i$ is the compressive strength of the equivalent strut based on the undamaged wall, $M_p$ is the flexural capacity of the column (or the lower value as provided by the beam plastic moments), $a$ is the gap opening length due to infill wall crushing, $H_w$ is the wall height, $\theta$ is the inclined angle of the strut, and $\alpha$ is the strut force reduction factor.

For the column mechanism, the remaining wall is stronger than the column. The undamaged portion of the infill wall restricts the deformation of the column and activating a captive column failure (Fig. 2b). The column shear demand of the column in this case ($V_b$) can be estimated using Eq. $V_b = \frac{2M_p}{a}$ (2).

$$V_b = \frac{2M_p}{a} \tag{2}$$

The lower shear force obtained from Eqs. $V_a = \frac{2M_p}{H_w} + \frac{\alpha C_i \cos\theta (H_w - a)}{H_w}$ (1) and $V_b = \frac{2M_p}{a}$ (2) can be used as the required shear strength of the column:

$$V_u = \min(V_a, V_b) \tag{3}$$




(a) (b)

Fig. 2 – Failure mechanism: (a) Column-Infill Mechanism and (b) Column Mechanism [2]

The reduction factor was developed using the results from FEM analysis of RC frame with infill wall [2]. The following equation was proposed for the reduction factor.

$$\begin{array}{lll} \alpha = 1.0 & for & drift < 0.5\% \\ \alpha = -0.6(drift) + 1.3 & for & 0.50\% \le drift \le 1.50\% \\ \alpha = 0.4 & for & 1.50\% < drift \end{array} \tag{4}$$

and, in terms of the gap opening, $a$;

$$\begin{array}{lll} \alpha = 1.0 & for & a/H_w < 0.05 \\ \alpha = -1.1(a/H_w) + 1.05 & for & 0.05 \le a/H_w \le 0.6 \\ \alpha = 0.4 & for & 0.5 < a/H_w \end{array} \tag{5}$$

Once the shear demand has been established, the shear force in the column can be checked again the column capacity. The shear capacity depends on the shear span ($a/d$) which, in turn, depends on the dimension of unrestrained region ($a$). For a small gap opening, a strut and tie model is a suitable approach for predicting column shear strength [15]. The column shear strength in this case is usually controlled by compressive failure of the strut. As the drift increased, with the gap opening larger than 4 times the column depth, the shear strength of the column can be estimated by code-based formulas.

The design steps essentially involve checking the shear demand versus the shear strength for a given opening length ($a$). The process is repeated by increasing the gap opening dimension until it covers the entire length of the column. The details of the method are summarized elsewhere [2].

## 3. Study Frames and Analytical Model

To investigate the capability of the design strategy, a typical low-rise commercial building commonly found in Thailand was considered and its seismic performance was assessed using nonlinear static and dynamic analyses. A simplified macro model, using the equivalent compression struts concept, was used. The study frames and analytical model descriptions can be summarized as follows.

### 3.1 Study Frames

In the present study, a masonry infilled reinforced concrete (RC) building consisted of 4-story, 8- and 3-bay in long and short directions (denoted as x and y, respectively) structure was selected as a prototype. The typical story was 3 m in height and 4 m in span length. For the base-story, the pier with 1 m in height was used to connect between the foundation and the first floor as illustrated in Fig. 3. Masonry infill walls, with and without opening, were used as the partitions of the building. This building was assumed to be in an area with a low-seismicity level in Thailand. In this area, the design spectra at 0.2 second ($S_s$) and 1 second periods ($S_1$) are 0.434g and 0.122g, respectively. Two different design methods were used. First, the building was analyzed as it was a bare frame, without any consideration of infill wall following a conventional design approach. The frame was designed as Intermediate Moment Resisting RC Frames (IMRFs) following the ACI318-14 (2014) design standard [16]. This building is denoted as B1 building. For the second case, the column design method as described in Section 2 was applied to the B1 building for assessing the column's shear capacity due to wall-frame interaction. If insufficient column shear strength was observed, the redesign process was utilized on those columns. This building was denoted as B2. To increase the column shear strength, the concrete compressive strength was first increased. If necessary, the column size and reinforcement bars were subsequently changed. It should be noted that this process was applied to the column in all stories. Even though, the high column shear demands are normally concentrated only in the first few stories. Based on the shear demand estimation, all exterior columns in the B1 building must be redesigned to increase the shear strength. The most critical columns are the ones located at the corners and the along the long edges of the building, especially for those in the first story. The shear demand to capacity ratios of 1.19 and 1.24 were observed in the corner and edge columns, respectively, whereas the shear demand to capacity ratios of 1.01 was observed in the short-edge columns. The nominal yield strength ($f_y$) of the longitudinal and transverse reinforcement bars was assumed as 400 MPa. The concrete with compressive strength ($f'_c$) of 21 MPa was used for the B1 frame and 24 MPa for B2 frame. The masonry infill wall with an average compressive strength of 6.5 MPa was used in both cases.



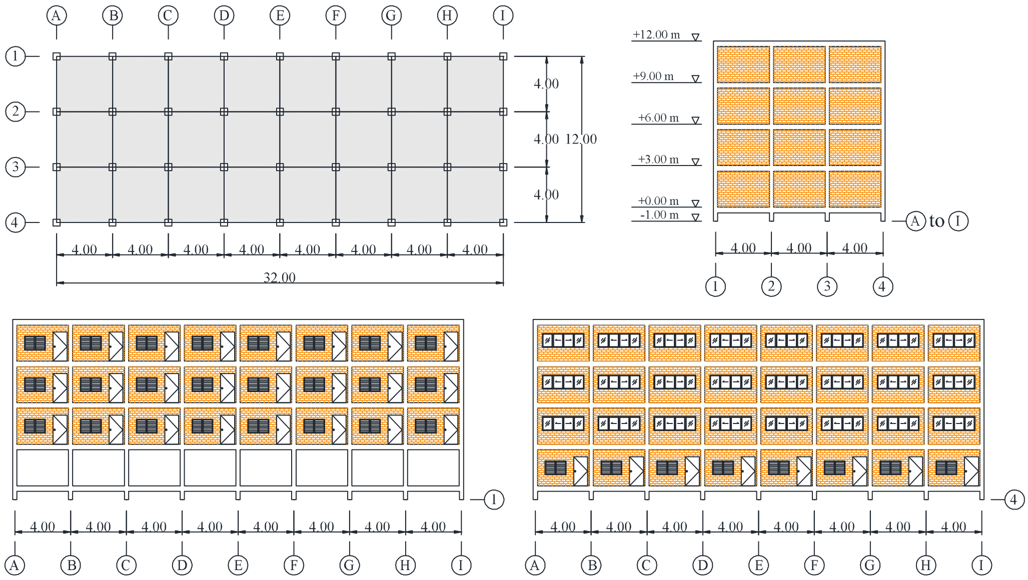



Fig. 3 – Masonryy infilled RC building used in this study

### 3.2 Analytical Model

The seismic performance assessment of the study buildings was performed using the 3-dimensional analytical model as showed in Fig. 4. The model was implemented in the commercial structural analysis software SeismoStruct (2020) [17]. The building slab was assumed a rigid diaphragm and the full restraint was utilized at the footings. The columns and beams were modeled using a force-based, fiber-section, element with five integration points. Concrete and reinforcement bars were represented by a uniaxial material. The well-known material constitutive models, Mander and Giuffre-Menegotto-Pinto models, were utilized for concrete and reinforcement bars, respectively. It should be noted that the shear failure of the RC members was not explicitly considered in the model for preventing numerical instability problem. The shear demand from the analysis was post-checked with the shear capacity to indicate shear failure. For the masonry infill wall, a simplified macro model, using the equivalent compression struts concept, was used. To consider a local effect on the columns due to infill wall interaction, the infill walls were modeled using the multi-strut model. The compression only 2-strut model proposed by Crisafulli and Carr (2007) [18] was adopted. The struts were oriented with one end located at the corner and the other end located at the distance equal to $2d$ from the top or bottom face of the beam, where $d$ is a column's effective width as illustrated in Fig. 4. The struts were modeled by fiber-section truss elements. A uniaxial material with Kent-Scott-Park constitutive model was utilized. The values of 0.0025 and 0.006 were adopted for strains at peak ($\varepsilon_m$) and at residual point ($\varepsilon_u$), respectively. The residual strength ($f_r$) was assumed as 0.1 times the masonry compressive strength ($f'_m$). According to the study result reported by Wararuksajja et al. (2020) [2], the column shear demand strongly depends on the strength of the damaged infill strut, with the strength ranging between 0.4-0.7 times that of the undamaged one, depending on structural response levels. Based on this observation, the strut width was assumed to be equal to $0.7w$ and $0.3w$ for the lower and upper struts, respectively, where $w$ is the effective width of the total strut estimated by the formula proposed by Tucker (2007) [9], as shown in Eq. $w = 0.25 d_w (\lambda H)^{-0.4}$ (6). This formula was conducted based on an extensive literature review. To simplify the calculation, all possible failure modes of the infill wall are combined into a single equation. In this method, the effective width of the equivalent strut is given by

$$w = 0.25 d_w (\lambda H)^{-0.4} \tag{6}$$



where $d_w$ is the diagonal length of the infill, $H$ is the height of the frame, and $\lambda$ is the relative stiffness of the surrounding frame and masonry infill wall proposed by Stafford and Carter (1962) [19] and is given by

$$\lambda = \sqrt[4]{\frac{E_m t_w \sin 2\theta}{4EIH_w}} \tag{7}$$

The initial strut strength of the solid infill wall was calculated as

$$C_i = \psi f'_m w t_w \tag{8}$$

where $E_m$ and $E$ are the modulus of elasticity of infill wall and concrete, respectively, $t_w$ is the wall thickness, $I$ is the moment of inertia of the column, and $\psi$ is a numerical factor depending on the masonry type. In the present study, the value of 1.90 is specified for the $\psi$. For the infill wall with opening, the strut strength is reduced by applying the reduction factor on Eq. $C_i = \psi f'_m w t_w$ (8). This reduction factor was estimated through finite element analysis of the infilled frame.

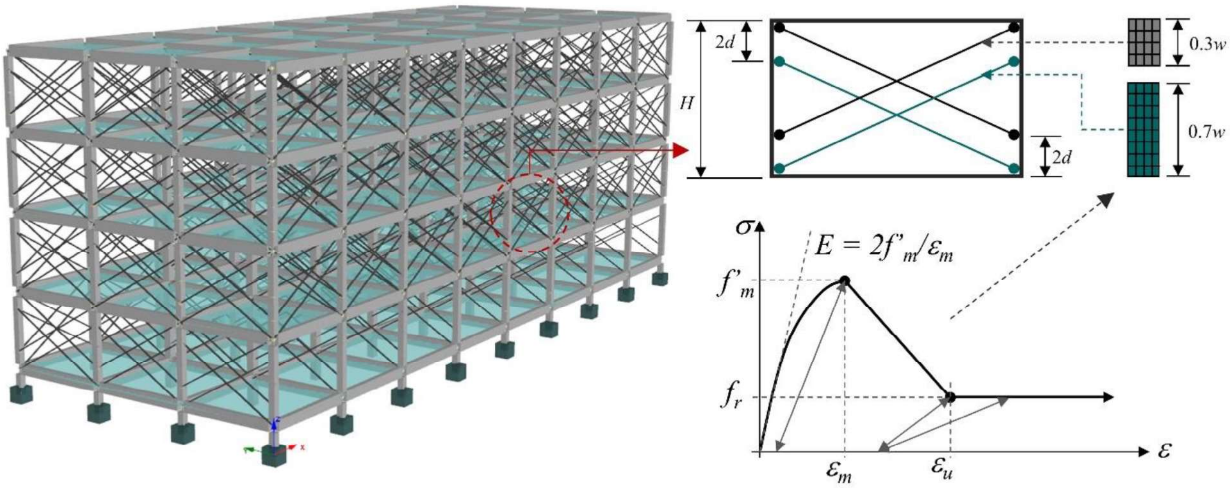



Fig. 4 – Analytical model of the study building

## 4. Performance Evaluation

The performance of the prototype buildings designed following both design strategies was evaluated through the nonlinear response history analysis and pushover analysis. The analytical condition and obtained key results of those analyses are discussed herein.

### 4.1 Nonlinear response history analysis

The nonlinear response history analyses (NLRHA) were utilized to investigate the performance of the study buildings under earthquake action. In the present study, Rayleigh damping with a specified damping ratio of 5% for the first and last modes of interest was used. Seven pairs of ground motion records were selected. Those records were scaled so that their spectra matched with the maximum considered earthquake (MCE) spectrum according to Thai seismic design code [20][21] as illustrated in Fig. 5. Each pair of the ground motions was applied in the horizontal direction. The vertical ground motion was neglected in the analysis.



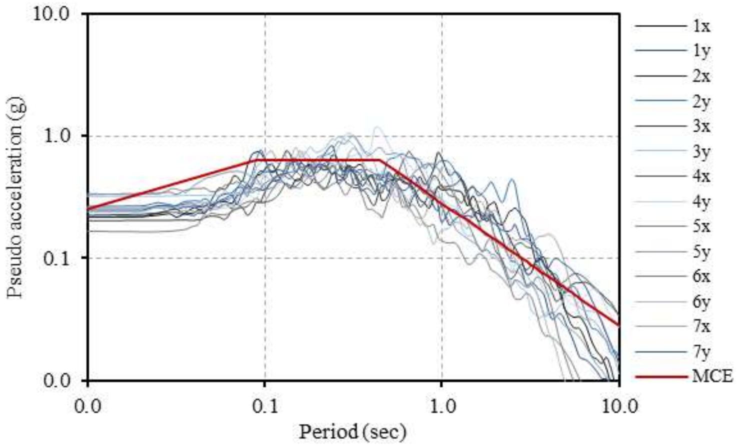



Fig. 5 – Maximum considered earthquake spectrum and spectra of the selected ground motions

Under the considered ground motions, the envelopes of the story drifts in both orthogonal directions of the B1 building are depicted in Fig. 6. The results show that the maximum and average values of first story drift in the x-direction were 2.01% and 1.33%, whereas the corresponding values in the y-direction were approximately 0.88 % and 0.40%, respectively. The average story drift in the x-direction was considerably higher than that one in the y-direction. The difference arose from the amount of infill wall present in both directions, which differ significantly. Furthermore, the deformation in the x-direction concentrated on the first and second stories resulting from the infill wall damaged. However, the column shear failure was not observed in this direction. For the y-direction, the deformation tended to distribute over the building height for all ground motions except ground motion no. 4. Under this ground motion, the infill strut forces induced the columns shear failure in the first story as indicated in Fig. 7. However, only the corner columns suffered this failure.

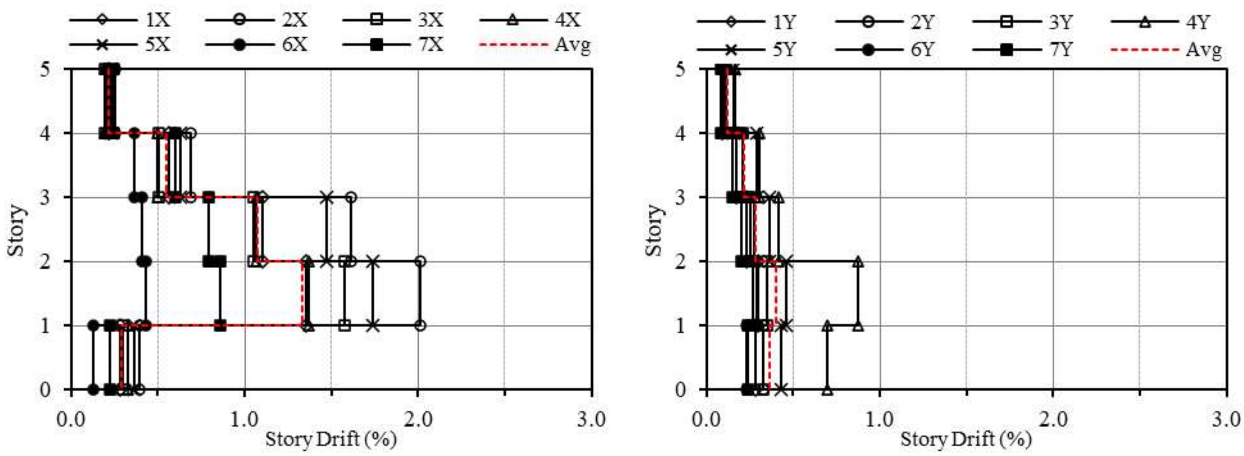



Fig. 6 – Inter-story drift envelopes of B1 building



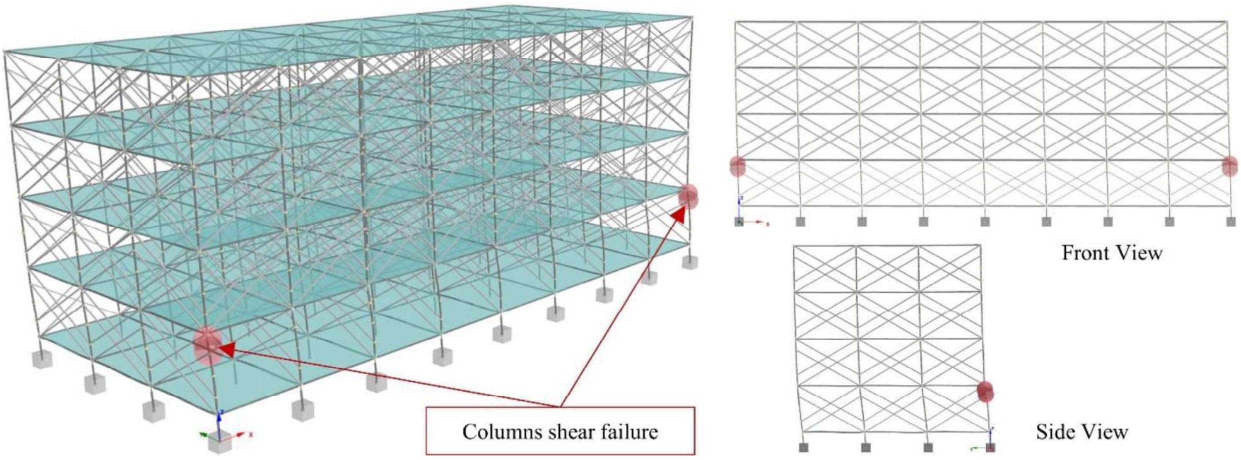


Fig. 7 – Columns shear failure of B1 building

The response of the B2 building was also evaluated through NLRHA. According to failure mechanism of the B1 building, the columns shear failure only occurred when the building was subjected to ground motions no. 4. Hence, only ground motions no.4 was considered in the analysis of B2 building. The envelopes of the story drifts in x- and y-directions of the B1 and B2 buildings are compared in Fig. 8. The overall response of those buildings were similar. The maximum story drift occurred in the first story for both directions. For the B2 building, the story drifts of 1.25% and 0.94% could be observed in the x- and y-directions, respectively. Unlike in building B1, the column shear failure was not observed in this case.

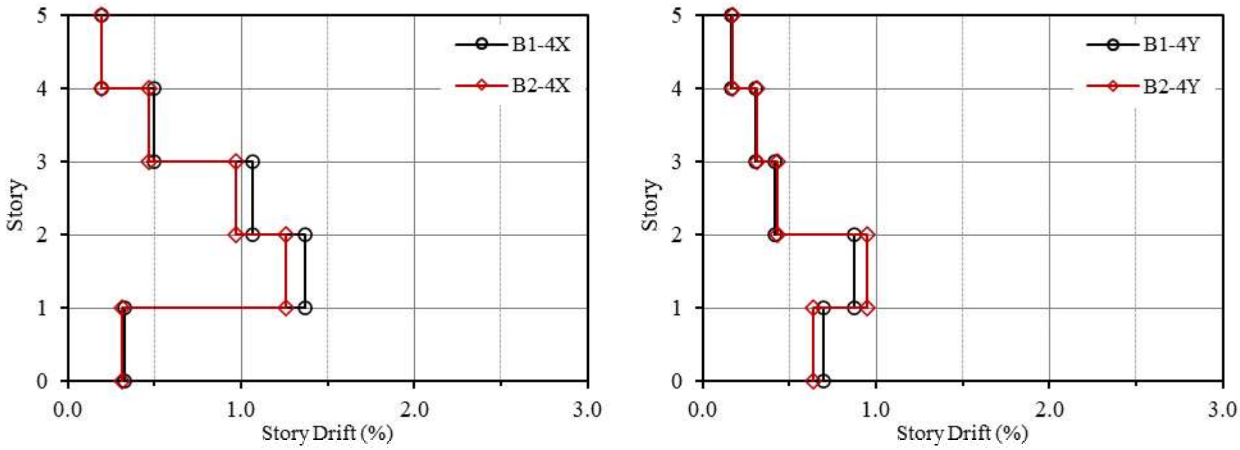


Fig. 8 – Comparison of inter-story drift envelopes of the study buildings

## 4.2 Nonlinear static analysis

The nonlinear pushover analyses were carried out to gain further insight in the response. The main objective was to investigate the column failure mechanism of the study buildings. Especially, when the building was excited by the ground motions stronger than those considered previously. Based on the NLRHA results, the columns shear failure was likely for the response in the y-direction of the building. Therefore, only this direction was considered in the pushover analyses. The lateral loading consistent with the first vibration mode was applied. Fig. 9 shows the pushover curves of the B1 and B2 buildings. The B1 building reached its peak resistance of 6508 kN at the roof drift of 0.40% corresponding to the first story drift of 0.60% as indicated in Fig. 9a. After this point, the load suddenly dropped due to the infill struts damage and the first story drift increased rapidly as a result. Similar to the NLRHA results, the infill strut forces induced the columns shear failure in the first story of the B1 building as indicated in Fig. 10. The corner and long-edge columns failed. This failure mechanism was consistent with the prediction using the design method discussed earlier. For the B2 building, the overall response was similar to that of the B1 building with a slightly higher peak resistance. However, the key difference arose in terms of column failure, where no column shear failure was detected in the B2 building.

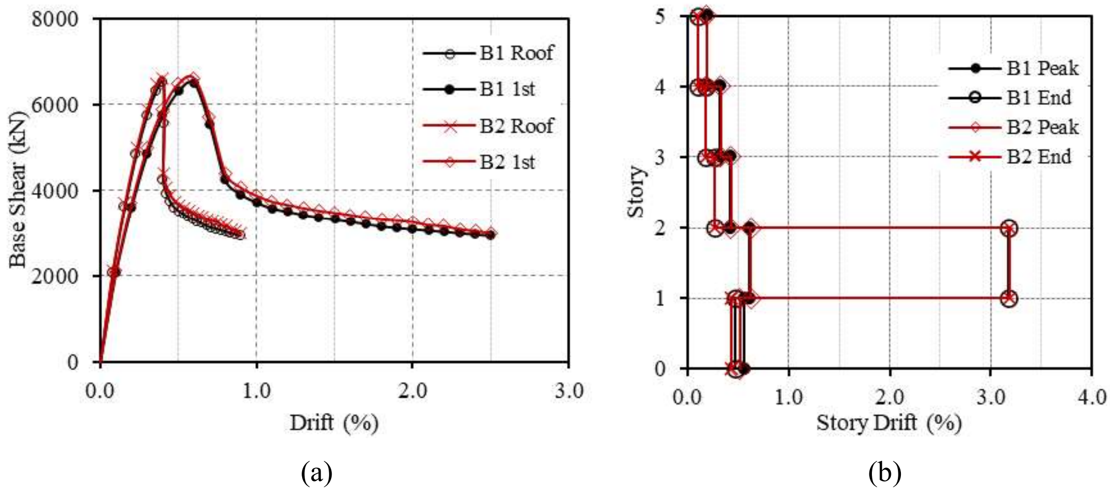



Fig. 9 – Response of study buildings; (a) Pushover curve and (b) Story drift

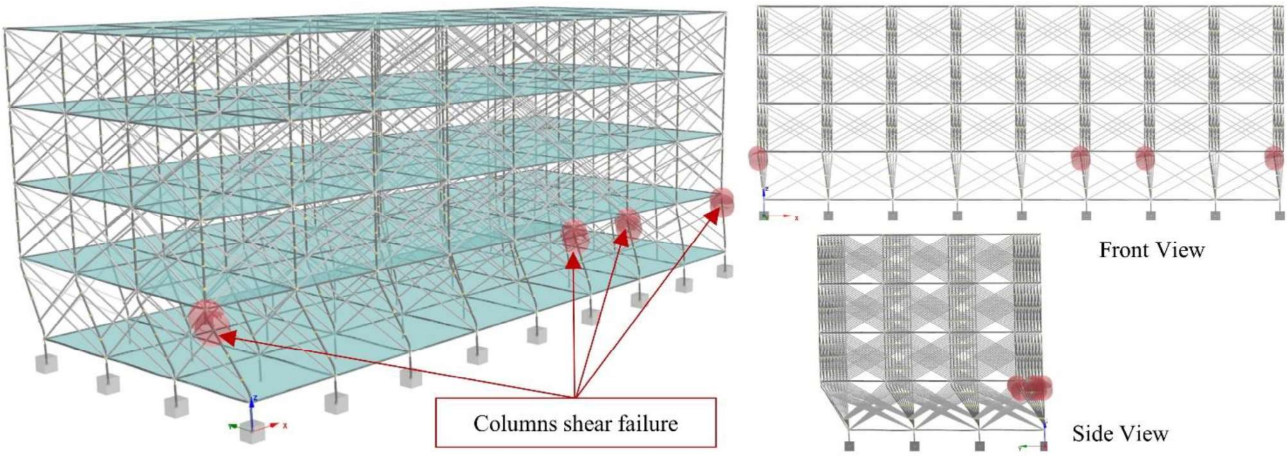



Fig. 10 – Columns shear failure of B1 building obtained by pushover analysis

## 5. Conclusions

In the present study, an effective design strategy to consider infill-frame interaction is investigated. The main concept for the design approach is briefly reviewed. The approach was applied to design a prototype RC building with infill walls located in an area with a low-seismicity level in Thailand. The performance of these buildings was assessed through nonlinear static and dynamic analyses. The failure modes and the effectiveness of the proposed design approach are also discussed.

Based on the study results, the overall response of the building designed with and without consideration of infill walls was similar. Only minor differences in the peak load resistance were observed. However, the columns shear failure occurred in the building designed without consideration of the infill walls but did not occur in the counterpart building. The local shear failure of the columns predicted by the method recently proposed by Wararuksajja et al. (2020) [2] shows good agreement with the analytical results. More importantly, by applying this design method, the column shear failure is eliminated. However, the method may provide an overestimation of column shear demand, especially on the upper part of the building. Further study is still needed in this aspect. In addition, the accuracy of the design method strongly depends on the infill strut capacity. Therefore, the infill strut capacity should be estimated by an appropriate approach and the calibrating process with locally available materials is a necessary part.

## 5. Acknowledgement

The authors gratefully acknowledge the financial support from the Thailand Research Funds (RDG6130022) and the King Mongkut's University of Technology Thonburi's Petchra Pra Jom Klao Ph.D. Scholarship to the first author. Supplementary funding was provided by TRF Senior Research Scholar under Grant RTA 6280012. The opinions presented are those of the authors alone and may not necessarily reflect the views of the sponsors.

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