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Numerical Investigation on the Progressive Collapse Resistance of an RC Building with Brick Infills under Column Loss

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***Abstract*—**Interior brick-infill partitions are usually considered as non-structural components and only their weight is accounted for in practical structural design. In this study, their effect on the progressive collapse resistance of an RC building subjected to sudden column loss is investigated. Three notional column loss conditions with four different brick-infill locations are considered. Column-loss response analyses of the RC building with and without brick infills are carried out. Analysis results indicate that the collapse resistance is only slightly influenced by the brick infills due to their brittle failure characteristic. Even so, they may help to reduce the inelastic displacement response under column loss. For practical engineering, it is reasonably conservative to only consider the weight of brick-infill partitions in the structural analysis.

***Keywords*—**Progressive collapse, column loss, brick-infill partition, compression strut.

I. Introduction

PROGRESSIVE collapse is referred to the phenomenon of widespread propagation of structural failure initiated by local damage. Many practitioners and academic researchers have been engaged in the prevention of progressive collapse since the partial collapse of the Ronan Point apartment building in 1968. Resistance of building structures to progressive collapse has become an important task for the development of structural design codes. It is learned from the history that unexpected abnormal loading is usually the ringleader for a progressive collapse event. Fortunately, the occurrence of a fatal accidental loading is extremely rare as compared to code-specified design loadings. Even so, reliable and simple approaches are undoubtedly required for evaluating the progressive collapse potential of important structures. Detailed step-by-step, linear static analysis procedures have been issued by the US General Service Administration (GSA) [1] and Department of Defense (DoD) [2]. The GSA linear static analysis approach has been applied to evaluate the progressive collapse potential of steel moment frames and RC frames [3-5]. Several analytical studies regarding the progressive collapse potential have been performed recently [6-9]. A static unloading experiment has been carried out to simulate the column loss and collapse-resistant behavior of a 1/3-scaled RC frame [10]. The experimental results indicate that the tested frame experienced elastic, plastic, and catenary phases during the progressive collapse process.

For conventional RC buildings, the brick-infill panels are usually adopted for interior partitions. They are often considered as non-structural elements and only their weight is approximately accounted for in practical structural design. Several studies have shown that the brick infills may contribute to the horizontal seismic resistance of an RC frame [11-15]. The brick-infill panels are usually modeled by equivalent compression struts [16-19]. A recent study indicates that steel bracings may be used to reduce the flexural demand of an RC building subjected to corner column loss [20]. Hence, it may be necessary to clarify the mechanical effect of brick infills on the progressive collapse resistance of RC buildings. Tsai and Huang [21] have investigated the influence of brick infills on the elastic response of an RC building under column loss. The brick infills may contribute to reduce the elastic column-loss response. Extended from the previous study, the influence of brick-infill partitions on the progressive collapse resistance of the RC building is examined in this paper. Equivalent compression struts are used to simulate the brick-infill panels. Three column loss conditions with four different brick-infill locations are considered. Column-loss responses of the RC building with and without brick infills are investigated to clarify their effectiveness in the plastic phase.

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II. RC Building Model and Column Removal

A. Structural frame model

The RC building is a 10-story, moment-resisting frame structure with a 2-story basement. Its first story is an open space for the public. As shown in Fig. 1, there are three bays with center-to-center span length arranged as 7.15m, 9.95m, and 7.15m in the longitudinal (west-east) direction, and two bays with a 5.48m and a 7.87m span in the transverse (north-south) direction. The story height is 4m for the first story and 3.3m for the others. In addition to the self weight, a dead load (DL) of 0.98kN/m^2 is applied to the roof and 0.245kN/m^2 to other floors. The service live load (LL) is 4.91kN/m^2 for the roof and 1.96kN/m^2 for other floors. Table 1 presents the section

dimensions of the RC members for the building. A compressive strength equal to 27500kN/m^2 is used for the concrete. The design yield strength is 412000kN/m^2 for the main reinforcements and 275000kN/m^2 for the stirrups.

TABLE I SECTION DIMENSIONS OF THE RC MEMBERS

Floor	Column	Peripheral beam	Interior beam	Joist
1F	70×100	60×90	50×90	30×65
2F	70×100, 70×90	60×75	50×75	30×65
3~4F	70×90	60×75	50×75	30×65
5~10F	70×90	50×75	50×75	30×65

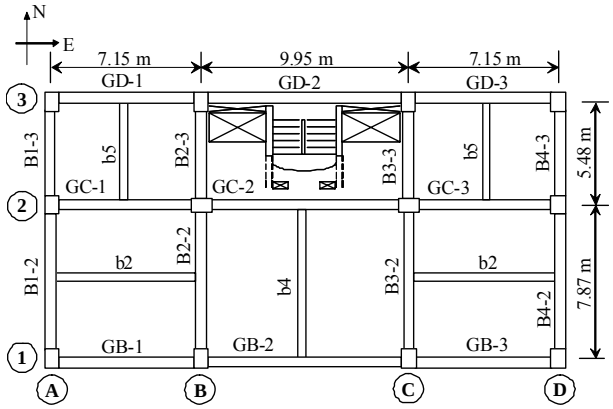

Fig. 1 Plan dimensions of the building

The building is located at a soft soil site and its design spectral response acceleration, S_{aD}, is equal to 0.45g. All the beams and columns are designed and detailed according to the seismic code requirements in Taiwan. A beam-column frame model is constructed for the RC building using the SAP2000 commercial program [22]. The building model is fixed on the ground. Self weight of the slabs and all the dead loads and live load on them are distributed to the beam elements for each floor. The fundamental period of the building model is equal to 1.48 and 1.40 seconds in the longitudinal and transverse direction, respectively.

Flexural plastic hinges are assigned to both ends of beam elements. Default moment-hinge properties based on the FEMA-356 guidelines [18] are adopted for the hinge model. Sectional moment capacity is calculated according to the design drawing. Preliminary analyses that collapse of the column-removed building is governed by the flexural failure mode of beam elements. The column members remain elastic when the joint beam sections have developed their ultimate moment capacities. Hence, shear failure is not considered and the column members are assumed to be elastic. Since the major concern is the difference between the column-loss response with and without the brick infills, the catenary action is not considered in the analysis.

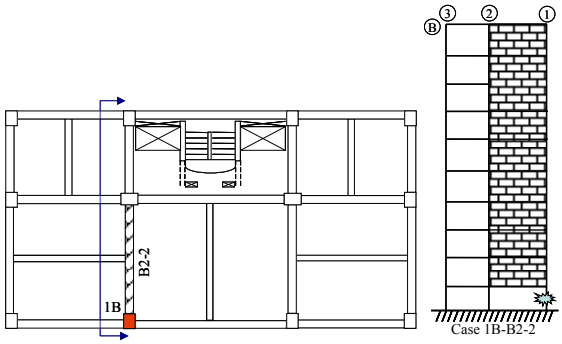

Fig. 2(a) Location of brick infill in Case 1B

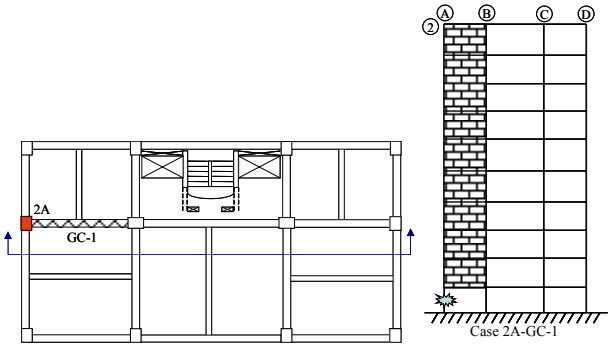

Fig. 2(b) Location of brick infill in Case 2A

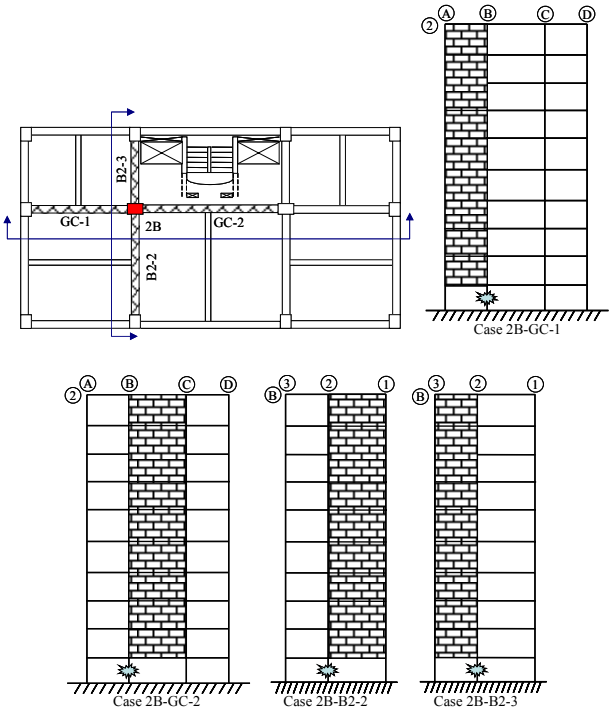

Fig. 2(c) Locations of brick infill in Case 2B

B. Column loss scenario

Three threat-independent, column-loss conditions, designated as Case 1B, Case 2A, and Case 2B, are considered for the building. According to the bay line numbers in Fig. 1, the removed column of the first story is 1B, 2A, and 2B for Case 1B, 2A, and 2B, respectively. For each column-removed

condition, the brick-infill panel is filled from the second to the top story of an interior structural bay next to the removed column. Because of the open space requirement, no brick infill is provided in the first story. Four different locations are considered to investigate the influence of the brick-infill layout, as shown in Figs. 2(a) to 2(c), where each hatched area indicates an analysis case. The brick infills are only filled in one bay next to the removed column in each case. Numbering of brick infills is given by its corresponding beam number. The designation of analysis cases with brick infill is provided by a combination of the column-removed case and the brick-infill number.

III. Equivalent Compression Strut

A. Modified FEMA model

From some studies of brick-infill RC frames subjected to horizontal loadings, the brick panels are usually modeled by compression-strut elements [11-14, 16-19]. A key issue in the strut modeling is the determination of equivalent strut width. In the FEMA-306 and FEMA-356 guidelines, the equivalent width for horizontal seismic analysis, denoted by a, is estimated as [18, 19]

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf}, \quad \lambda_1 = \left[\frac{E_{me} t_{inf} \sin 2\theta}{4 E_{fe} I_{col} H_{inf}}\right]^{1/4} \qquad (1a, 1b)$$

where h_{col} and r_{inf} are the column height between centerlines of beams and the diagonal length of infill panel, respectively. t_{inf} is the thickness of panel and strut. H_{inf} and E_{me} are respectively the height 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 height-to-length aspect ratio in radian. However, the loading direction of a building frame under sudden column loss is quite different from horizontal seismic excitation. As shown in Fig.3, it is vertical downward loading imposed on the brick-infill panel as the building loses a supporting column. Therefore, the estimation of the strut width may be reasonably modified as

$$a = 0.175(\lambda_1 L_b)^{-0.4} r_{inf}, \quad \lambda_1 = \left[\frac{E_{me} t_{inf} \sin 2\phi}{4 E_{fe} I_b L_{inf}}\right]^{1/4} \qquad (2a, 2b)$$

where L_b is the beam length between centerlines of columns, I_b is the moment of inertia of beam. ϕ is the angle whose tangent is the infill length-to-height (L_{inf} / H_{inf}) aspect ratio in radian. L_{inf} is the horizontal length of infill panel. As recommended by FEMA 356, E_{me} is calculated as $550 f_m'$, where f_m' is the compressive strength of the infill and assumed as 4142kPa in this study.

B. Paulay's model

A conservative suggestion for the strut width, which may induce potentially higher seismic response, is provided by Paulay and Priestley [23]. It is recommended that

$$a = 0.25 r_{inf} \qquad (3)$$

Although this formula does not explicitly consider the effect of the peripheral confinement provided by columns or beams, it is quite practical for engineering application. Axial and vertical compressive stiffness of the strut may be respectively expressed by

$$k_a = E_{me} a t_{inf} / r_{inf}, \text{ and } k_v = k_a \sin^2 \theta \qquad (4a, 4b)$$

Apparently, based on Eq.(3), axial stiffness of the equivalent strut is independent of the diagonal length of the brick infill.

Table 2 summarizes the mechanical properties of the compression strut estimated by the two equivalent width models for the four infill positions. The thickness of struts is 12cm, which is equal to that of the brick infills. It is seen that different models lead to varied estimations. Hence, they are used to examine the influence of strut modeling on the column-loss response of a brick-infilled RC building.

TABLE II ELASTIC PROPERTIES OF THE EQUIVALENT COMPRESSION STRUTS

Model	Location	GC-1	GC-2	B2-2	B2-3
Modified FEMA-356 (Eq.2)	ϕ [a]	68.12	74.26	70.17	61.42
	k (kN/m)[b]	32560	30372	31833	34688
	a (m)	0.82	1.05	0.88	0.68
Paulay and Priestley (Eq.3)	k (kN/m)	68343	68343	68343	68343
	a (m)	1.71	2.35	1.88	1.33

a: unit in degree, k: axial stiffness of the strut

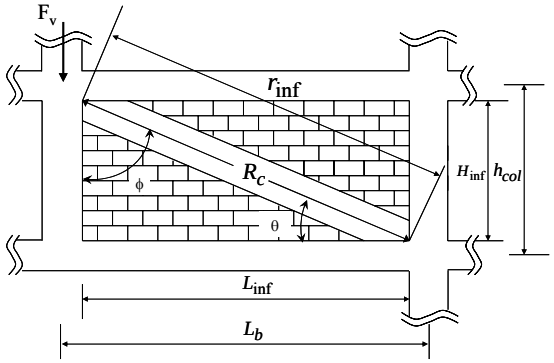

Fig. 3 Loading direction of the brick infill

C. Plastic strut model

As observed from several experimental studies [14, 16, 24], the strength capacity of an infill panel is a complex phenomenon. Since the brick-infill panel of a column-removed building is subjected to a downward loading, it is presumed that the strut strength is not controlled by the sliding-shear or general shear failure modes, which are dominated by the shear capacity of bed joints. Meanwhile, it is pointed out that the diagonal tensile cracking of the infill panel does not really constitute a failure condition, since higher horizontal force may be resisted by the diagonal compression failure mode [23]. Hence, it is assumed that the brick panel loaded by a column loss condition is dominated by diagonal compression failure, and its compressive strength, R_c, is expressed as [19, 25]

$$R_c = at_{\inf} f'_{me90} \quad (5)$$

where f'_{me90} is the horizontal expected strength of infill panel and calculated as 50% f'_{me}. f'_{me} is the expected compressive strength of test brick prism and estimated as 1.3 f_m'.

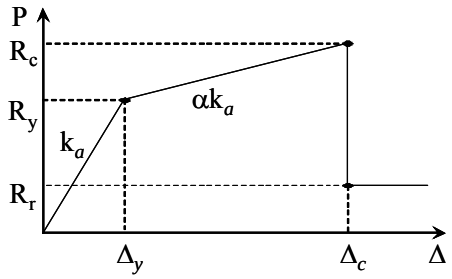

Fig. 4 Nonlinear compressive model of the equivalent strut

TABLE III NONLINEAR PARAMETERS OF THE EQUIVALENT COMPRESSION STRUTS

Model	Modified FEMA-356			
Item	GC-1	GC-2	B2-2	B2-3
Δ_y (cm)	-0.54	-0.74	-0.59	-0.41
R_y (kN)	-175	-226	-187	-142
Δ_c (cm)	-1.9	-2.6	-2.1	-1.5
R_c (kN)	-264	-338	-284	-218
Model	Paulay and Priestley			
Item	GC-1	GC-2	B2-2	B2-3
Δ_y (cm)	-0.53	-0.72	-0.58	-0.41
R_y (kN)	-359	-495	-395	-280
Δ_c (cm)	-1.9	-2.6	-2.1	-1.5
R_c (kN)	-552	-755	-604	-427

A multi-linear model is used to simulate the nonlinear behavior of the equivalent strut [11-12, 16], as shown in Fig.4. Since the nonlinear brick infill is assumed as a compression-only element, the plastic model cannot resist tensile force. Its ultimate compressive strength is estimated by Eq.(5). The corresponding displacement response, Δ_c, is determined by [16-17]

$$\Delta_c = \varepsilon'_m r_{\inf} \quad (6)$$

where ε'_m is the compressive strain at the maximum compressive stress and estimated by [24]

$$\varepsilon'_m = C_j \frac{f'_m}{E_{me}^{0.7}},\ C_j = \frac{0.27}{f_j^{0.25}} \quad (7a, 7b)$$

In the above equation, f_j is the compressive strength of mortar and equal to 9.8 MPa in this study [26]. The post-stiffness ratio, α, is assumed as 0.2. Therefore, the equivalent compressive yielding force R_y and its corresponding displacement Δ_y may be respectively determined by

$$R_y = \frac{(R_c - \alpha k_a \Delta_c)}{(1-\alpha)} \text{ and } \Delta_y = R_y / k_a \quad (8a, 8b)$$

$R_r = 0.3R_y$ is assumed for the residual strength of the equivalent strut to account for the interface friction force after cracking [11]. The calculated nonlinear parameters for the considered brick-infill cases are presented in Table 3.

IV. PROGRESSIVE COLLAPSE RESISTANCE

A. Nonlinear static response

Displacement-controlled nonlinear pushdown analysis is carried out under the loading pattern of (DL+0.25LL) applied to the adjacent bays of the failed column. Displacement of the column-removed point is adopted as the representative of overall deformation. Figs. 5(a) and 5(b) show the load-displacement responses of Case 1B and Case 2A conditions, respectively. Due to the brittle failure characteristic, contribution of the brick infills to plastic strength is only effective on the initial yielding phase. As expected, larger strut width incurs more increased strength. Similar responses are obtained for the Case 2B conditions, as shown in Figs. 6(a)~6(d). It is learned from Eq.(6) that the larger the $L_{\inf} / H_{\inf}$ ratio of brick-infill panels, the larger the ultimate displacement capacity. Hence, the brick-infill panels of Case 2B-GC-2 failed at a larger displacement than the others.

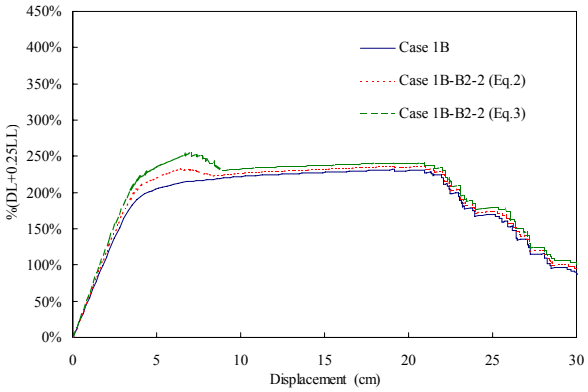

Fig. 5(a) The load-displacement responses of Case 1B-B2-2

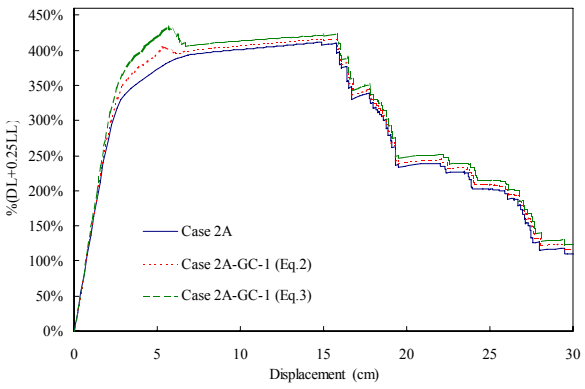

Fig. 5(b) The load-displacement responses of Case 2A-GC-1

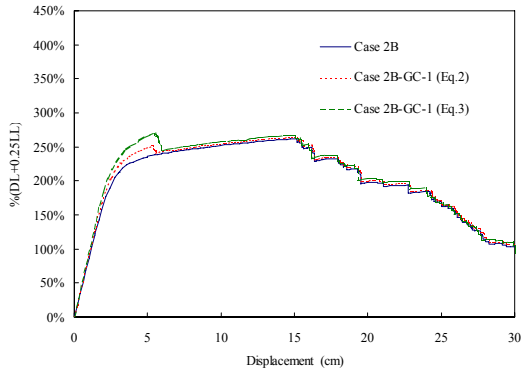

Fig. 6(a) The load-displacement responses of Case 2B-GC-1

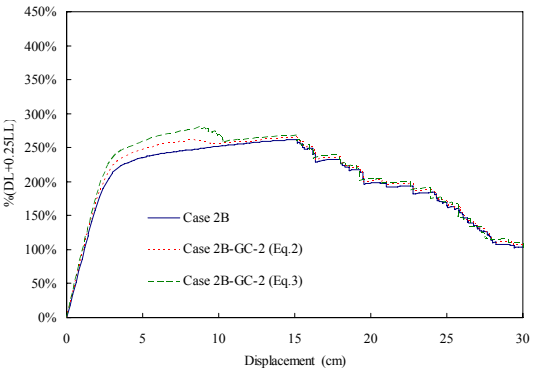

Fig. 6(b) The load-displacement responses of Case 2B-GC-2

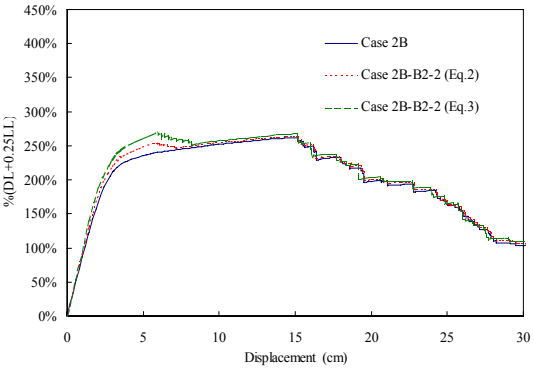

Fig. 6(c) The load-displacement responses of Case 2B-B2-2

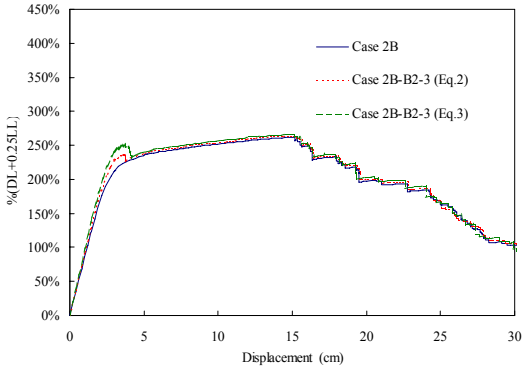

Fig. 6(d) The load-displacement responses of Case 2B-B2-3

B. *Collapse resistance estimation*

From the nonlinear static load-displacement response, it appears that the brick infills only slightly increase the collapse resistance under column loss. This may be confirmed from the pseudo-static response curves of the column-loss building. A pseudo-static curve is established on the principle of equal work and strain energy and defined as

$$P_{CC}(u_d) = \frac{1}{u_d}\int_0^{u_d} P_{NS}(u)du \tag{9}$$

$P_{CC}(u_d)$ and $P_{NS}(u)$ are respectively the pseudo-static and the nonlinear static loadings estimated at the displacement demand u_d and u. In fact, it is obtained from the nonlinear static load-displacement curve. The maximum value of the pseudo-static curve may be used to approximate the collapse resistance of the column-removed building [5, 8].

Fig. 7(a) and 7(b) show the pseudo-static response curves under the column-loss scenarios of Case 1B and 2A, respectively. It is seen that although the increased resistance is observable, but not significant. Since the brick infills usually fail prior to the ultimate strength of the frame, the resistance increment mainly comes from the residual strength of the failed brick infills. The strut model with larger equivalent width may induce higher resistance. Similar results are obtained for the column-loss scenarios of Case 2B, as shown in Figs. 8(a)~8(d). Consistent with the nonlinear static results, the brick infills with a larger length-to-height ratio may have more observable resistance increment, such as Case 2B-GC-2. Table 4 summarizes the estimated collapse resistance in terms of (DL+0.25LL) for the column-loss building without and with brick infills. It is seen that both the modified FEMA and Paulay's model have similar collapse resistance.

TABLE IV COLLAPSE RESISTANCE WITH AND WITHOUT INFILLS

Case	No infills	With infills	$L_{\text{inf}} / H_{\text{inf}}$	Modified FEMA-356	Paulay and Priestley
1B	2.02	1B-B2-2	2.77	2.09	2.17
2A	3.56	2A-GC-1	2.49	3.66	3.76
2B	2.25	2B-GC-1	2.49	2.30	2.35
		2B-GC-2	3.55	2.32	2.40
		2B-B2-2	2.77	2.30	2.36
		2B-B2-3	1.84	2.29	2.33

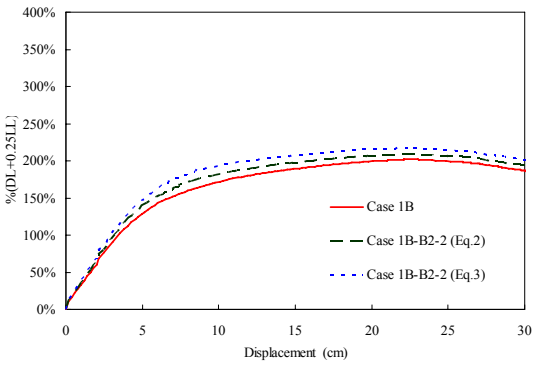

Fig. 7(a) Pseudo-static response curves of Case 1B-B2-2

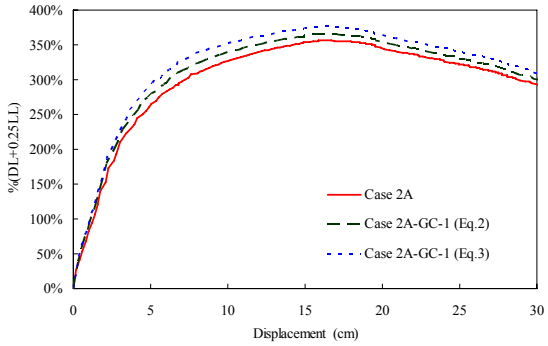

Fig. 7(b) Pseudo-static response curves of Case 2A-GC-1

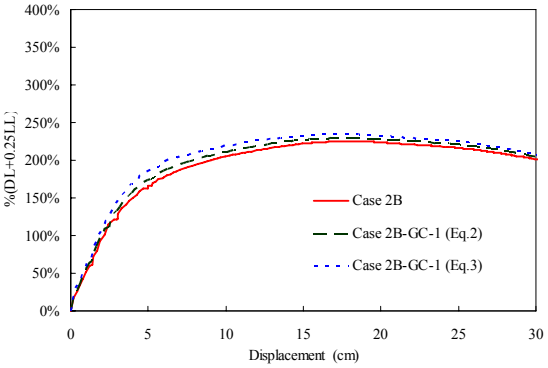

Fig. 8(a) Pseudo-static response curves of Case 2B-GC-1

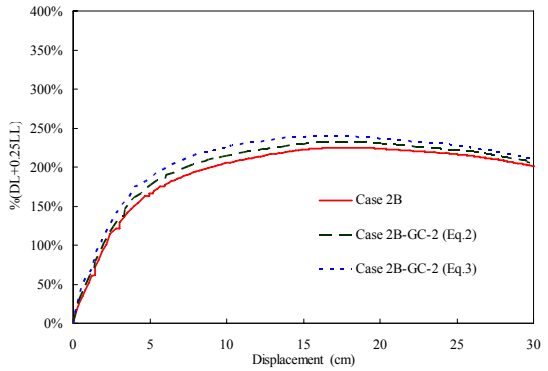

Fig. 8(b) Pseudo-static response curves of Case 2B-GC-2

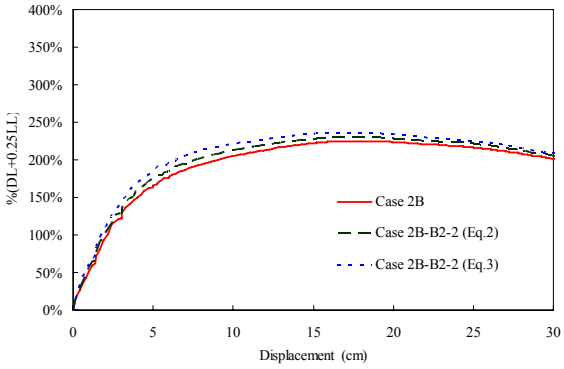

Fig. 8(c) Pseudo-static response curves of Case 2B-B2-2

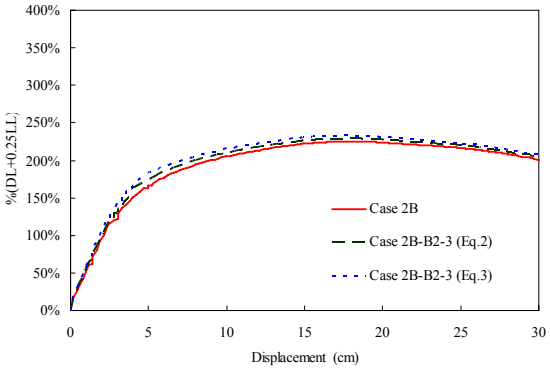

Fig. 8(d) Pseudo-static response curves of Case 2B-B2-3

C. *Nonlinear dynamic response*

For a given applied downward loading, it is observed from the pseudo-static curves that the dynamic displacement under column loss may be reduced with the brick infills, even though they are insignificant for the collapse resistance. Nonlinear time history analysis is thus carried out for Case 2B to examine the extent of displacement reduction. A downward loading of 2.2(DL+0.25LL), which is close to the collapse resistance of Case 2B, is statically applied to the column-removed building. Meanwhile, a set of concentrated loading, which is equal to the internal sectional force of the failed column, is also imposed at the column-removed point to simulate the intact condition. Then, a set of equal-but-opposite loading is suddenly applied to the column-removed point for simulating the column loss scenario. The whole procedure is illustrated in Fig. 9. Figs. 10(a) and 10(b) show the dynamic displacement responses with brick infills simulated by the modified FEMA and Paulay's models, respectively. It is seen that the displacement response is indeed reduced with the brick-infill panels. Table 5 summarizes the maximum dynamic displacement and the associated percentage of reduction. Due to larger deformation and loading capacities, the Paulay's strut model leads to lesser displacement response.

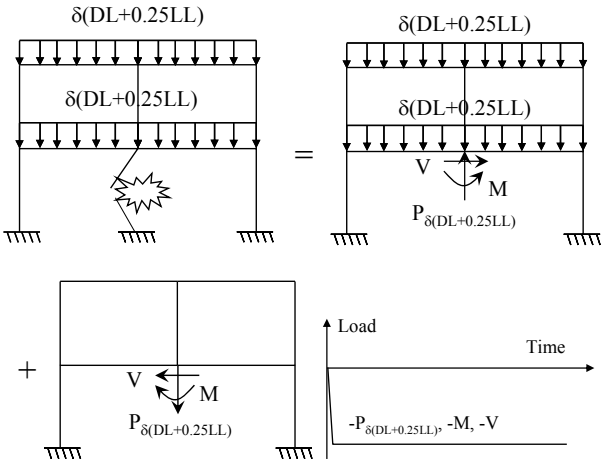

Fig. 9 Dynamic simulation of sudden column loss

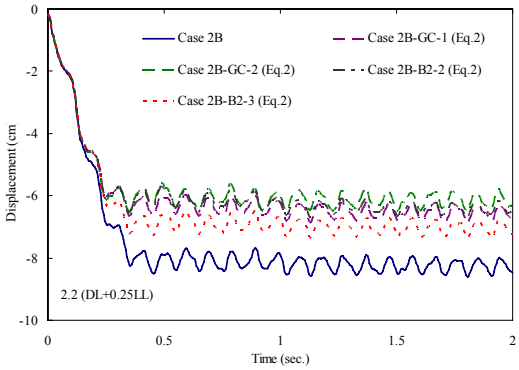

Fig. 10(a) Nonlinear dynamic displacement histories of Case 2B (modified FEMA model)

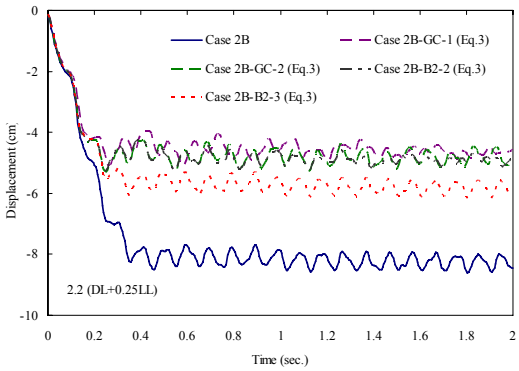

Fig. 10(b) Nonlinear dynamic displacement histories of Case 2B (Paulay's model)

TABLE V MAXIMUM DYNAMIC DISPLACEMENT AND PERCENTAGE OF REDUCTION UNDER 2.2(DL+0.25LL)

Case	No infills	With infills	Modified FEMA-356	Paulay and Priestley
2B	8.6	2B-GC-1	6.9 (20%)	5.1 (41%)
		2B-GC-2	6.6 (23%)	5.3 (38%)
		2B-B2-2	6.7 (22%)	5.3 (38%)
		2B-B2-3	7.4 (14%)	6.2 (28%)

V. CONCLUSIONS

Effect of interior brick-infill partitions on the plastic responses of a moment-resisting RC building under column loss is evaluated in this study. Compression-strut elements are used to simulate the brick infills and the strut width is estimated by two different models. Based on the plastic brick-infill model, a larger L_{inf} / H_{inf} ratio gives delayed brittle failure. Nonlinear dynamic analysis results confirm that the brick infills may reduce inelastic displacement response under column loss. Even so, the collapse resistance of the column-removed building is not significantly changed due to the brittle failure characteristic of brick infills. For practical engineering, it may be reasonably conservative to consider the weight of brick-infill partitions without accounting for their stiffness and strength.

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