World Academy of Science, Engineering and Technology
International Journal of Civil and Environmental Engineering
Vol:9, No:2, 2015

Seismic Performance of Reinforced Concrete Frames Infilled by Masonry Walls with Different Heights

Ji–Wook Mauk, Yu–Suk Kim, Hyung–Joon Kim

***Abstract*—**This study carried out comparative seismic performance of reinforced concrete frames infilled by masonry walls with different heights. Partial and fully infilled reinforced concrete frames were modeled for the research objectives and the analysis model for a bare reinforced concrete frame was also established for comparison. Non–linear static analyses for the studied frames were performed to investigate their structural behavior under extreme seismic loads and to find out their collapse mechanism. It was observed from analysis results that the strengths of the partial infilled reinforced concrete frames are increased and their ductilities are reduced, as infilled masonry walls are higher. Especially, reinforced concrete frames with higher partial infilled masonry walls would experience shear failures. Non–linear dynamic analyses using 10 earthquake records show that the bare and fully infilled reinforced concrete frame present stable collapse mechanism while the reinforced concrete frames with partially infilled masonry walls collapse in more brittle manner due to short-column effects.

***Keywords*—**Fully infilled RC frame, partially infilled RC frame, masonry wall, short–column effects.

I. Introduction

INFILLED masonry walls have been long recognized as an effective element that can resist to strong ground motion since it provides additional stiffness and strength to a lateral force-resisting system. The lateral force resisting mechanism of reinforced concrete (RC) structures infilled by masonry walls is complicate because of the interactions between RC members and masonry infills [1], [2]. The seismic performance of RC framed buildings infilled by masonry infills has been investigated and started to initiate to be adopted in structural codes [3], [4]. The paper evaluates the seismic performances of RC frames infilled by masonry walls with different heights throughout non–linear static and dynamic analyses. Also, the effects of infilled masonry wall's heights on the collapse mechanism of structures are studied in detail.

Ji-Wook Mauk, Graduate Student, is with the Department of Architectural Engineering, University of Seoul, Siripdae-Gil 13, Dongdaemun-Gu, Seoul, 130-743, Republic of Korea (e-mail: ahrwldnr@uos.ac.kr).

Yu-Suk Kim, Ph.D Student, is with the Department of Architectural Engineering, Kwangwoon University, Gwangun-ro 20, Nowon-Gu, Seoul, 139-701, Republic of Korea (email: archimac2010@gmail.com)

Hyung-Joon Kim, Associate Professor, is with the Department of Architectural Engineering, University of Seoul, Siripdae-Gil 13, Dongdaemun-Gu, Seoul, 130-743, Republic of Korea (corresponding author to provide phone: 82-2-2210-2743; fax: 82-2-2210-4835; e-mail: hyungjoonkim@uos.ac.kr).

II. Non–Linear Static Analysis

A. Analytical Models

To observe interactions between a RC frame and masonry infills, 2.66 m wide and 2.10 m tall structures introduced in [1], [13] were considered as the prototype frames. In this study, columns were designed with horizontal reinforcements spaced by every 150 mm to simulate short–column effects.

As shown in Fig. 1, the ratios of infilled heights (H_i) to the story height (H_s) were considered as a main analysis parameter ranging from 0.0 to 1.0 with an interval of 0.25. It is noted that H_{00} and H_{100} frames, respectively, represent bare and fully infilled frames and frames of H_{25}, H_{50} and H_{75} are partially infilled frames where subscriptions denote the percentile ratios.

The efforts to establish reliable macro–models have been lasted for many decades. Modeling schemes for masonry infills using diagonal struts, equivalent frames, equivalent panels and finite elements have been suggested by researchers. Recently, Equivalent strut models have been recognized as useful and reliable tools to estimate seismic responses of infilled frames [5]–[9], [15]–[18].

In this study, masonry walls of the prototype structures were modeled using a modeling scheme proposed by [8], [9]. As shown in Figs. 2 (a), (b), the scheme is to use both equivalent struts and equivalent panels for diagonal compression and shear sliding of masonry infills. RC members of the structures were modeled using frame elements.

The flexural behavior of RC members was evaluated from *Response2000* [10] and the shear behavior of RC members was evaluated to capture short–column effects from *FEMA–356* [7]. Parameters required for the hysteresis of masonry walls were referred to [1], [6] and [7]. A computer software, *Ruaumoko2D* [11] was used for non–linear analyses.

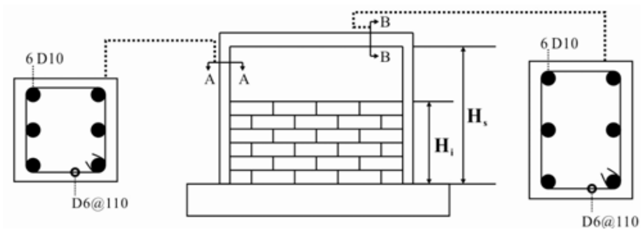
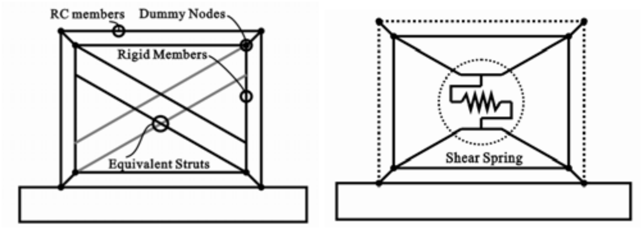


Fig. 1 Configuration of prototype frames



(a) Equivalent struts (b) Equivalent panel

Fig. 2 Modeling scheme proposed by *Crisafulli* [1]

B. Non–Linear Static Analyses

The collapse mechanism of the prototype structures has been investigated using non–linear static analyses. As shown in Fig. 3 and Table I, the increase in the ratio of H_i/H_s increases maximum base shear, V_{max} while it decreases the maximum story drift, Δ_{max}. However, the prototype structure, H_{100} develops the largest loading carrying capacity with largest deformation capacity.

From important events shown in Fig. 3 and Table I, the prototype structures of H_{00} and H_{25} collapse in the flexural failure mechanism observed in typical RC frames while other structures of H_{50} and H_{75} fracture in brittle manner due to short–column effects. In contrast to the partial infilled frames, the fully infilled frame of H_{100} shows most stable behavior.

Cyclic push–over analyses were performed to investigate their collapse mechanisms and to confirm reliabilities of analytical models under dynamic loadings. A loading pattern used in the experiments tested by *Crisafulli* [1] is imposed to the analysis models. The cyclic responses of the models are different with those obtained from monotonic loading. Their deformation capacities assessed by the cyclic push–over analyses are generally overestimated except for the analysis model of H_{100} of which the cyclic response is enveloped by the monotonic behavior. Figs. 4 (a)–(e) present the cyclic push-over analysis results. As shown in the figures, the model structures of H_{00} and H_{25} show ductile collapse mechanism due to flexural failures of RC members while other structures of H_{50} and H_{75} show brittle collapse mechanism due to short–column effects resulting from the shear failure of RC column members. Especially, the model structure of H_{75} with a relatively high masonry infill shows fatal failures of RC columns.

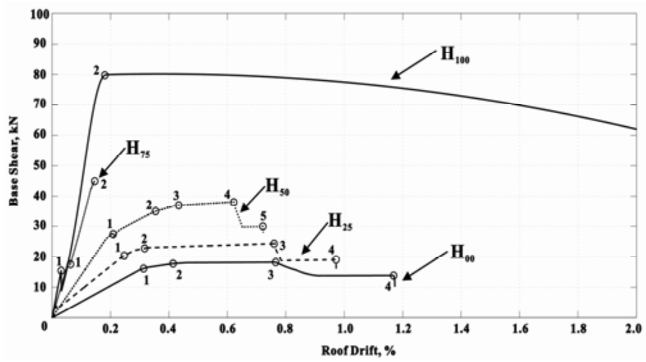
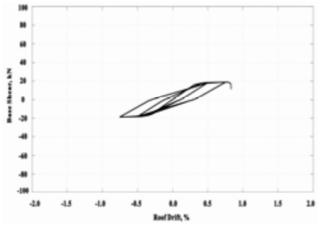
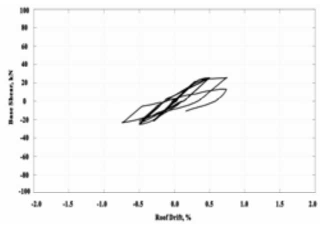


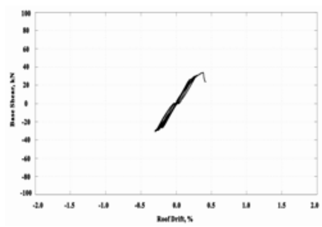
Fig. 3 Comparisons of push–over analysis results



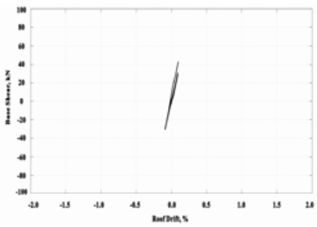
(a) H_{00} Case



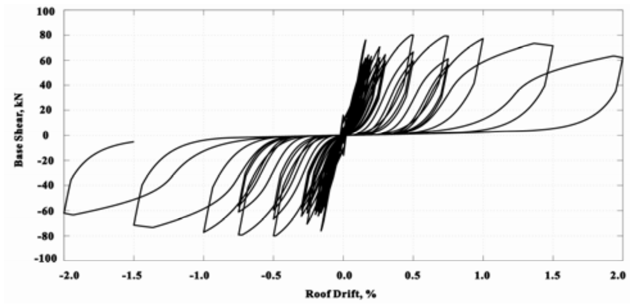
(b) H_{25} Case



(c) H_{50} Case



(d) H_{75} Case



(e) H_{100} Case

Fig. 4 Cyclic push–over analysis results

TABLE I
RECORDS OF PUSH-OVER ANALYSIS

Records	Descriptions				
	H_{00}	H_{25}	H_{50}	H_{75}	H_{100}
1	Right column yielded	Right column yielded	Panel sliding & Struts crushed	Panel sliding	Panel sliding
2	Left column yielded	Left column yielded	Right column yielded	Left column failed for short–column effects	Struts crushed
3	Degradation of right columns starts	Degradation of right columns starts	Left column yielded	–	–
4	Right column failed	Right column failed	Degradation of right columns starts	–	–
5	–	–	Right column failed	–	–
V_{max}	18.2kN	24.3kN	38.1kN	44.9kN	80.2kN
Δ_{max}	1.15%	0.96%	0.71%	0.15%	2.00%

Such brittle mode behavior is more clearly presented as the partial masonry infills are getting higher. From these observations, the introduction of the capacity design concept is required for the design of RC columns unbraced by a partial masonry infill in order to prevent brittle failure modes. Sufficient shear reinforcements are designed so that the flexural dominant behavior is governed. Unlike the bare RC frame and the partially infilled RC frames, the fully infilled frame of H_{100} shows the most ductile behavior under cyclic loading.

III. Non–Linear Dynamic Analysis

Non–linear dynamic analyses using 10 historical strong ground motion records were carried out to investigate the seismic response of the prototype structures. A suite of 10 far–field records consists of No. 1 to 10 records out of 44 records introduced in *FEMA–P695* [12], [14]. The records were normalized to remove unwarranted variability and were then scaled to the design spectrum with S_{DS}=1.0g and S_{D1}=0.6g representing design based earthquakes (DBEs). For the comparison of seismic responses of the prototype structures under maximum considered earthquakes (MCEs), the spectrum with S_{MS}=1.5g and S_{M1}=0.9g was also used to calibrate the records.

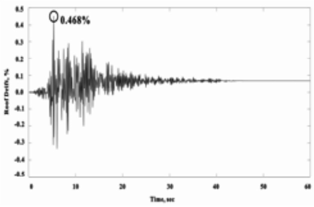
TABLE II
Median Story Drift Ratios Obtained from Dynamic Analyses

	Prototype Structures				
	H_{00}	H_{25}	H_{50}	H_{75}	H_{100}
$\Delta`_{max, DBE}$	0.43%	0.29%	0.15%	0.12%	0.02%
$\Delta`_{max, MCE}$	0.72%	0.44%	0.28%	0.16%	0.07%

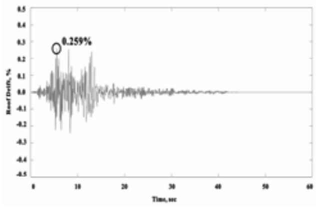
Median values of maximums story drift ratios ($\Delta`_{max, DBE}$, and $\Delta`_{max, MCE}$) of the prototype structures under both DBEs and MCEs are listed in Table II. Under both DBEs and MCEs, the structure of H_{00} shows the largest median value of $\Delta`_{max}$ while the smallest is developed in the structure of H_{100}. The increase in the height of masonry walls decreases the seismic deformation demands. This is due to the fact that the masonry walls improve the lateral resistance of the frame. The increase in the lateral stiffness due to the presence of the masonry walls presents negligible effects on the seismic demands because the prototype structures considered in this study are in the acceleration constant region.

Although median story drifts provide important information on the seismic response of brittle structures under strong ground motion, the number of collapse is also an important criterion since story drifts of collapse structures are considered as an outlier in a statistic point of view. Under the DBE seismic hazard level, it is noted that there is no earthquake where the structures of H_{25}, H_{50} and H_{100} collapse while the H_{00} and H_{75} structures collapse in one and five earthquakes, respectively. Under the MCE seismic hazard level, there are five earthquakes making the structures of H_{00} and H_{25} to collapse while the H_{50} and H_{75} structures collapse in seven earthquakes. Unlike the observation that the RC frames with higher masonry infills present better seismic story drift response, the bare RC frame and the lower masonry infilled RC frame have small collapse probabilities because of their more ductile seismic response. As expected, the fully infilled frame of H_{100} presents the smallest probability that it suffers the collapse under a single earthquake out of 10 records.

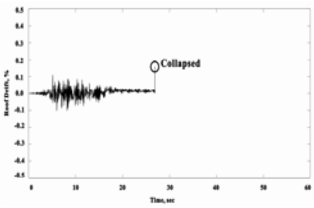
In order to investigate the seismic response of the prototype masonry infilled RC frames in detail, the results of time history analyses under the *Hector Mine* earthquake event calibrated to the MCE seismic hazard level are presented in Figs. 5 (a)–(e). As shown in the figures, the structure of H_{00} suffers structural damage without collapse while the H_{25} frame experiences negligible structural damage. The partial infilled frames of H_{50} and H_{75} are seismically collapsed in a brittle manner. The H_{50} and H_{75} frames totally lost their lateral load resistances after they reach to very small story drift demands. The structure of H_{100} seismically behaves in stable manner. Furthermore, it is noted that the structure of H_{00} has residual displacements while there is no residual displacements observed in the structures of H_{25} and H_{100}.



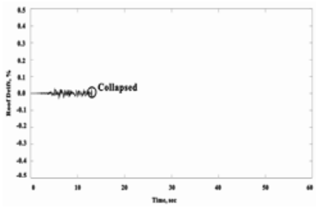
(a) H_{00} Case



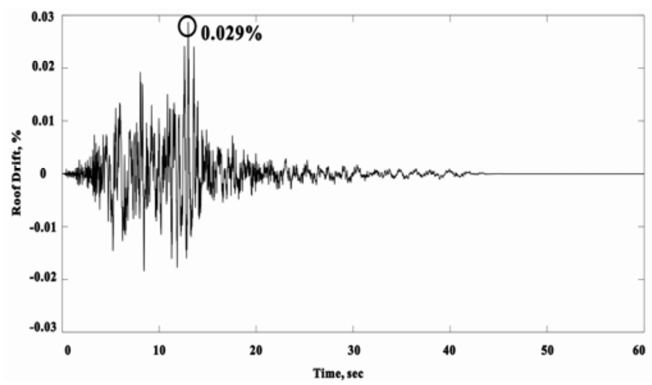
(b) H_{25} Case



(c) H_{50} Case



(d) H_{75} Case



(e) H_{100} Case

Fig. 5 Comparison of time history analysis results

IV. Conclusions

The paper evaluates the seismic performances of RC frames infilled by masonry walls with different heights throughout non–linear static and dynamic analyses. In non–linear static analyses, collapse capacities and collapse mechanism of the prototype structures have been investigated. In non–linear dynamic analyses, seismic responses under both DBE and MCE seismic hazard levels have been compared. The significant observations are summarized as follows:

(1) Non–linear static analyses of the prototype structures present that maximum base shear forces of the structures, H_{25}, H_{50}, H_{75}, and H_{100} are, respectively, 1.34, 2.09, 2.47, and 4.41 times as much as that of the structure, H_{00}. In contrast, maximum roof drift of the H_{25}, H_{50}, H_{75}, and H_{100} structures are 0.83, 0.62, 0.13, and 1.74 times as much as that of the H_{00} structure, respectively.

(2) While the collapse mechanism of the structure, H_{25} is similar to the flexural failure mechanism of the H_{00} structure, the structure of H_{75} collapses in a brittle manner due to short–column effects.

(3) From non–linear dynamic analyses, the median story drift of the H_{25} structure under the DBE level seismic hazard is 0.67 times as much as that of the H_{00} structure while the drift of the H_{25} structure under the MCE hazard level is 0.61 times as much as that of the H_{00} structure.

(4) From the dynamic analyses, the median story drift of the H_{75} structure under DBE level seismic hazard is 0.80 times as much as that of the H_{50} structure while the drift of the H_{75} structure under the MCE hazard is 0.57 times as much as that of the H_{50} structure.

(5) The structures of H_{25}, H_{50} and H_{100} do not collapsed under DBEs. However, the H_{00} and H_{75} structures, respectively, have 10% and 50% collapse probabilities with the same seismic hazard.

(6) The structures of H_{00} and H_{25} have 50% collapse probabilities under MCEs while the structures of H_{50} and H_{75} have 70% collapse probabilities under MCEs. In contrast, fully infilled frame of H_{100} has only 10% collapse probability with the same seismic hazard.

ACKNOWLEDGMENT

This research was supported by a grant 'Development of Socio-economic Seismic Loss Prediction Models' [NEMA-NH-2012-67] from the Natural Hazard Mitigation Research Group, National Emergency Management Agency of Korea.

REFERENCES

[1] F. J. Crisafulli, *Seismic behaviour of reinforced concrete structures with masonry infills*, University of Canterbury.
[2] S. Sattar, and A. B. Liel, "Seismic performance of reinforced concrete frame structures with and without masonry infill walls," *9th U. S. National and 10th Canadian Conference on Earthquake Engineering*, Toronto, Canada, 2010.
[3] ASCE/SEI 41-13, "Seismic evaluation and retrofit of existing buildings," *American Society of Civil Engineers*, Reston, Virginia, U. S. A.
[4] B. Tremayne, F. Turner, A. Russell, S. Oliver, and H. Derakhshan, "Proposed update to masonry provisions of ASCE/SEI 41: Seismic Evaluation and Retrofit of Exist Buildings," *WCEE World Conference*, 2012.
[5] P. G. Asteris, S. T. Antoniou, D. S. Sophiano poulos, and C. Z. Chrysostomou, "Mathematical macromodeling of infilled frames: State of the art," *the Journal of Structural Engineering*, vol. 137, no. 12, pp. 1508–1517, Dec. 2011.
[6] E. Smyrou, *Implementation and verification of a masonry panel model for nonlinear dynamic analysis of infilled RC frames*, Rose School.
[7] FEMA–356, "Prestandard and commentary for the seismic rehabilitation of buildings," *Federal Emergency Management Agency*, Washington, DC, Nov. 2000.
[8] F. J. Crisafulli, and A. J. Carr, "Proposed macro-model for the analysis of infilled frame structures," *Bulletin of the New Zealand Society for Earthquake Engineering*, vol. 40, no. 2, pp. 69–77, Jun. 2007.
[9] F. J. Crisafulli, A. J. Carr, and R. Park, "Anaytical modelling of infilled frame structures-A general review," *Bulletin of the New Zealand Society for Eathquake Engineering*, vol. 33, no. 1, pp. 30–47, Mar. 2000.
[10] C. Evan, Bentz, and M. P. Collins, "Response-2000: Reinforced Concrete Sectional Analysis Using the Modified Compression Field Theory," 1998.
[11] A. J. Carr, "RUAUMOKO–inelastic dynamic analysis program," *the Department of Civil Engineering, University of Canterbury*, Christchurch, New Zealand, 2008.
[12] FEMA–P695, "Quantification of building seismic performance factors," *Federal Emergency Management Agency*, Washington, DC, Jun. 2009.
[13] "Verification report for version 6," in *Seismostruct*, ch. 4, pp. 111–131.
[14] PEER, "PEER NGA database," *Pacific Earthquake Engineering Center*, University of California, Berkeley, U. S. A.
[15] V. K. R. Kodur, M. A. Erki, and J. H. P. Quenneville, "Seismic analysis of infilled frames," the Journal of Structural Engineering, vol. 25, no. 2, pp. 95–102, Jul. 1998.
[16] P. M. Pradhan, P. L. Pradhan, and R. K. Maskey, "A review on partial infilled frames under lateral loads," *Kathmandu University Journal of Science, Engineering and Technology*, vol. 8, no. 1, pp. 141–152, Feb. 2012.
[17] H. R. Tamboli, and U. N. Karadi, "Seismic Analysis of RC Frame Structure with and without Masonry Infill Walls," *Indian Journal of Natural Sciences*, vol. 3, no. 14, pp. 976–997, Sep. 2012.
[18] C. Dymiotis, A. J. Kappos, and M. K. Chryssanthopoulos, "Seismic reliability of masonry–infilled RC frames," *the Journal of Structural Engineering*, vol. 127, no. 3, pp. 296–305, Mar. 2001.