Computers and Concrete, *Vol. 22, No. 5 (2018) 439-447*
DOI: https://doi.org/10.12989/cac.2018.22.5.439

The investigation of seismic performance of existing RC buildings with and without infill walls

Hakan Dilmac[*1], Hakan Ulutas[2a], Hamide Tekeli[1b] and Fuat Demir[1c]

[1]Suleyman Demirel University, Department of Civil Engineering, Isparta, Turkey
[2]Burdur Mehmet Akif Ersoy University, Department of Civil Engineering, Burdur, Turkey

(Received July 6, 2018, Revised November 5, 2018, Accepted November 8, 2018)

Abstract. One of the important factors is the infill walls in the change of the structural rigidity, ductility, dynamic and static characteristics of the structures. The infill walls are not generally included in numerical analysis of reinforced concrete (RC) structural system due to lack of suitable theory and the difficulty of calculating the recommended models. In seismic regions worldwide, the residential structures are generally RC buildings with infill wall. Therefore, understanding the contribution of the infill walls to seismic performance of buildings may have a vital importance. This paper investigates the effects of infill walls on seismic performance of the existing RC residential buildings by considering requirements of the Turkish Earthquake Code (TEC). Seismic performance levels of residential RC buildings with and without walls in high-hazard zones were determined according to the nonlinear procedure given in the code. Pushover curves were obtained by considering the effect of masonry infill walls on seismic performance of RC buildings. The analysis results showed that the infill walls beneficially effected to the rigidity, roof displacements and seismic performance of the building.

Keywords: seismic performance; existing RC buildings; infill wall

1. Introduction

A large number of buildings are constructed with the masonry infills for architectural needs or aesthetic reasons. The behavior of masonry infilled frames has been extensively studied in the last four decades in attempts to develop a rational approach for design of such frames. An extensive review of research on infilled frames through the mid-1980's has been reported by Moghaddam *et al*. (1987). A number of past studies focused on evaluating the experimental behavior of masonry infilled frames to obtain formulations of limit strength and equivalent stiffness (Klingner *et al*. 1978, Bertero *et al*. 1983, Zarnic 1990, Mander *et al*. 1994, Madan 1997).

In recent years, structural damage control must also be taken into consideration in the design of earthquake resistant structure due to damages of buildings which caused serious pecuniary loss and spiritual damages. Accordingly, analyzes related to displacement-based design instead of design-based of buildings are become even more important (Poland *et al*. 1997). The nonlinear structural analyses are used to evaluate the earthquake behavior of structures with masonry infill walls (Atımtay 2000, 2001).

The contribution of infill walls has been realized on the structural responses of frames by many researchers (Reinhorn 1997, Nollet *et al*. 1998, Harpal *et al*. 1998, Sahota *et al*. 2001, Honget *et al*. 2002, Pujol *et al*. 2010, Sattar *et al*. 2010, Korkmaz *et al*. 2015, Dilmaç 2017, Bas *et al*. 2017). Similarly, some studies have been carried out to determine on seismic response of buildings with and without masonry infill wall using experimental evaluation, energy-based approach, probabilistic assessment or shaking-table test in order to develop efficient strengthening solutions in prevent collapse and improve their performance (Dolsek *et al*. 2008, Hermanns *et al*. 2014, Penna *et al*. 2014, Sattar *et al*. 2016, Furtado *et al*. 2016, Merter *et al*. 2017, Benavent-Climent *et al*. 2018, Peng *et al*. 2018).

The infill walls are generally considered as diagonal strut in the analytical models of buildings. In the last three decades, the studies have focused on developing diagonal strut model of infill wall. A method has been developed for the analysis of steel or reinforced concrete frames with masonry infill walls by Saneinejad and Hobbs (1995). The proposed method accounts for elastic and plastic behavior of infilled frames. The strength and stiffness of infilled frames as well as the diagonal cracking load can be predicted by the method.

The diagonal strut assumption for simulating the behavior of masonry infill wall has been found to be accurately sufficient in evaluating the response of masonry infilled RC framed buildings (Perera 2005, Samoil'a 2012, Asteris *et al*. 2012, Asteris *et al*. 2013, Kareem *et al*. 2018).The effect of the intensity and position of the infill walls with and without opening on the RC structures also affects the seismic behavior. The method proposed by

∗Corresponding author, Ph.D.
E-mail: hakandilmac@sdu.edu.tr
[a]Ph.D.
E-mail: hulutas@mehmetakif.edu.tr
[b]Ph.D.
E-mail: hamidetekeli@gmail.com
[c]Ph.D.
E-mail: fuatdemir@sdu.edu.tr

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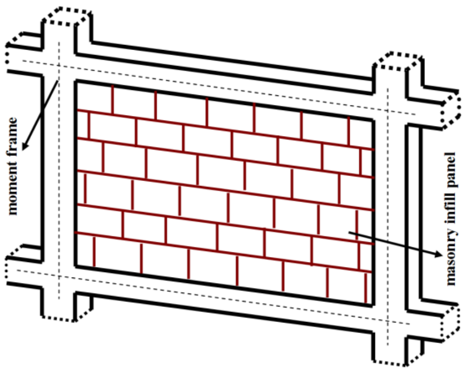
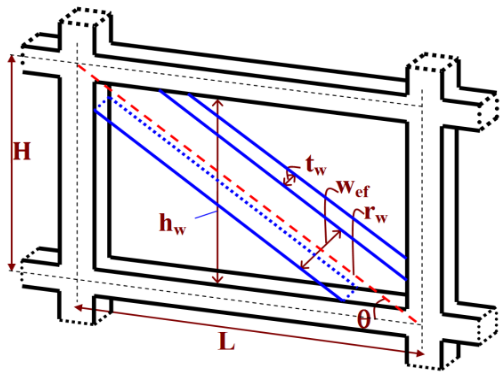
ISSN: 1598-8198 (Print), 1598-818X (Online)



(a)



(b)

Fig. 1 Models for masonry infill wall

Donaire-Avila *et al*. (2015) to determine intensity measures and predict the seismic behavior of frame structure with hysteretic dampers is based on the use of the response spectrum of a ground motion, such as spectral values and spectrum intensity parameters. The regularly distributed infill walls improve the shear capacity of buildings (Asteris 2017). In the other study by Asteris (2011) the damage effect of masonry infill wall on column and beam was examined to the scheme of the failure modes of in-filled frame with and without opening was presented and proposed different failure mode. The discontinuous and having plan irregularity infill walls may cause soft-storeys, torsion to the building and an increase of the shear at the adjoining columns. In additionally, the infills have cause to different changes in the most significant characteristic is the fundamental period of vibration (Asteris *et al*. 2015, 2016)

In this study, seismic performances of 120 existing residential buildings were investigated to the principles of nonlinear method given in the TEC (2007). The selected existing buildings in high-risk zone were analyzed by using SAP 2000 software program to obtain seismic performance level of buildings with and without masonry infill walls.

2. Modelling of masonry infill walls

The masonry infill walls can be modeled by using either a refined continuum model or equivalent strut model as given in Fig. 1(a)-(b), respectively. The former is simple, but it is theoretically weak. The modeling of equivalent nonlinear stiffness of the infill wall using diagonal struts is not simple, especially when there exist some openings in the wall. The latter method based on continuum model can provide an accurate computational representation of both material and geometry aspects, if the properties and the sources of nonlinearity of the masonry are carefully defined (Hao *et al*. 2002).

In this study, the infill wall is modeled using diagonal struts as given in Fig. 1(b). The width of infill wall is taken into account by Eq. (1).

$$w_{ef} = 0.175.(\lambda_w H)^{-0.4}.r_w$$

(TEC, 2007; FEMA, 356) (1)

where, w_{ef} is width of infill wall, H is height of story, r_w is diagonal length of infill wall and λ_w is stiffness factor. The stiffness factor of masonry infill wall is taken into account by Eq. (2).

$$\lambda_w = \left[\frac{E_w t_w \sin 2\theta}{4E_c I_c h_w}\right]^{0.25}$$

(TEC, 2007; FEMA, 356) (2)

where, t_w is thickness of infill wall, θ is angle of diagonal to horizontal in degrees, h_w is height of infill wall, L is length of span of equivalent diagonal strut, I_c is the moment of inertia of the column, and E_c and E_w are the elastic modulus of concrete and the infill wall, respectively. E_c and E_w are given in Eq. (3) and Eq. (4), respectively.

$$E_c = 5000\sqrt{f_{co}} \text{ (FEMA, 356)} \quad (3)$$

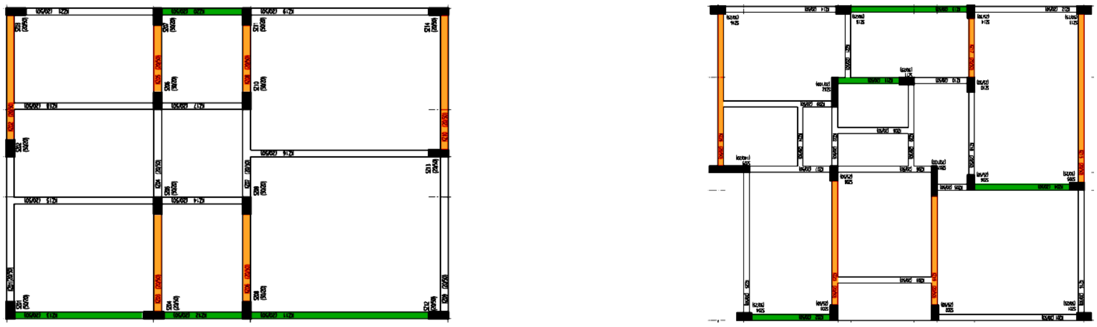
where f_{co} is the compressive strength of concrete in MPa

$$E_w = 1000MPa \quad \text{(TEC, 2007)} \quad (4)$$

3. Seismic performance of RC building

Seismic codes (TEC 2007, FEMA 356) propose using linear and nonlinear analysis methods to determine seismic performance of existing buildings. The nonlinear analysis is better than linear analysis to ensure the accurateness. The nonlinear static analyses require obtaining capacity curves of the RC buildings. In this reason, beam and column elements are modelled as nonlinear frame elements by defining plastic hinges at both ends of beams and columns. The plastic hinge length L_p is assumed to be half of the section depth. In the non-linear evaluation procedure, the pre-yield linear behavior of concrete sections is represented by cracked sections where their bending rigidity is assumed to be *0.40EIo* for beams and *(0.40-0.80) EIo* for columns depending on the level of the axial load, where EIo is the gross sectional bending rigidity (TEC 2007).

Concrete compressive strain and steel tensile strain demands at the plastic regions are calculated with the help of the moment-curvature diagrams at the plastic curvature level. Moment-curvature diagrams of the critical sections are obtained by applying appropriate stress-strain rules for concrete and steel. Unconfined and confined concrete models developed by Mander *et al*. (1988) are used in analyses.



(a) (b)

Fig. 2 The structural layouts of some existing RC buildings

Concrete compressive and steel tensile strains are used to determine damage states of structural members for performance assessment of a building according to TEC (2007). The code defines three damage limits as minimum damage limit (MN), safety limit (SL) and collapse limit (CL). The building earthquake performance levels are decided after determining the member damage levels. Four performance levels are defined for the building according to TEC (2007) that has similarities with FEMA-356 guidelines. The residential RC buildings are expected to satisfy the life safety performance level under the design spectrum obtained for %10 probability of exceeding in 50 years. The rules for determining building performance are given below for each performance level (TEC 2007):

Performance level defined as Immediate Occupancy (IO), in any story, in the direction of the applied earthquake loads, not more than 10% of beams are in the significant damage state whereas all other structural members are in the minimum damage state. Performance level defined as Life Safety (LS), in any story, in the direction of the applied earthquake loads, not more than 20% of beams and some columns are in the extreme damage state whereas all other structural members are in the minimum or significant damage states. However, shear carried by those columns in the extreme damage state should be less than 20% of the story shear at each story. Performance level defined as Collapse Prevention (CP), in any story, in the direction of the applied earthquake loads, not more than 20% of beams and some columns were in the collapse state whereas all other structural members are in the minimum, significant or extreme damage states. However, shear carried by those columns in the collapse state should be less than 20% of the story shear at each story. Furthermore, such columns should not lead to a stability loss. Occupancy of the building should not be permitted. Performance level defined as Collapse (C), if the building fails to satisfy any of the above performance levels, it is accepted as in the collapse state.

Table 1 The structural properties and the analysis results of some buildings

ID	n	Material	T_1 (s)	A_{ft} (m^2)	W_b (kN)	$d^{ep}{}_{max}$ (m)	$S_{d(ay)}$ (m)	R_y	μ
A-1	2	MGA	0.34	92	2000	46	0.028	1.24	1.33
		MGB	0.41			77	0.032	1.86	1.98
		MGAw	0.30			30	0.022	1.08	1.15
		MGBw	0.33			52	0.017	1.69	2.55
A-2	2	MGA	0.41	140	2510	89	0.023	2.34	3.07
		MGB	0.43			105	0.019	3.87	4.47
		MGAw	0.31			52	0.017	1.66	2.50
		MGBw	0.33			76	0.008	4.09	8.12
B-1	3	MGA	0.67	1040	23600	182	0.042	3.51	3.27
		MGB	0.69			193	0.046	5.02	3.21
		MGAw	0.59			152	0.043	2.63	2.66
		MGBw	0.63			174	0.033	4.97	4.05
B-2	3	MGA	0.61	153	4929	150	0.055	2.15	2.11
		MGB	0.64			174	0.037	4.68	3.62
		MGAw	0.49			92	0.041	1.64	0.98
		MGBw	0.51			101	0.032	4.36	3.69
C-1	4	MGA	0.75	199	7337	204	0.048	3.51	3.33
		MGB	0.76			209	0.027	6.02	6.05
		MGAw	0.50			106	0.042	1.71	1.96
		MGBw	0.52			121	0.035	2.33	2.76
C-2	4	MGA	0.72	229	11600	193	0.058	2.98	2.42
		MGB	0.74			205	0.054	4.88	2.77
		MGAw	0.61			152	0.046	2.28	2.45
		MGBw	0.65			173	0.037	3.47	3.43
D-1	5	MGA	1.00	413	25077	310	0.081	4.01	2.79
		MGB	1.02			254	0.054	4.96	3.46
		MGAw	0.93			283	0.081	3.39	2.51
		MGBw	0.99			307	0.043	6.86	5.29
D-2	5	MGA	1.02	376	23520	319	0.055	5.08	3.48
		MGB	1.09			351	0.057	8.08	4.16
		MGAw	0.87			260	0.053	3.09	2.41
		MGBw	0.89			193	0.051	8.12	4.12

4. Determining seismic performance of existing buildings

The major portion of the building stock in many developing countries are consists of deficient low and midrise reinforced concrete buildings (Ozmen *et al*. 2015). In this study, the seismic performance level of existing mid-rise reinforced concrete buildings were investigated by using pushover analysis procedure. The 120 existing RC buildings located in high-hazard zones in Turkey were selected in order to investigate seismic performance of building considering nonlinear behavior of reinforced concrete components as well as masonry infill walls. In the

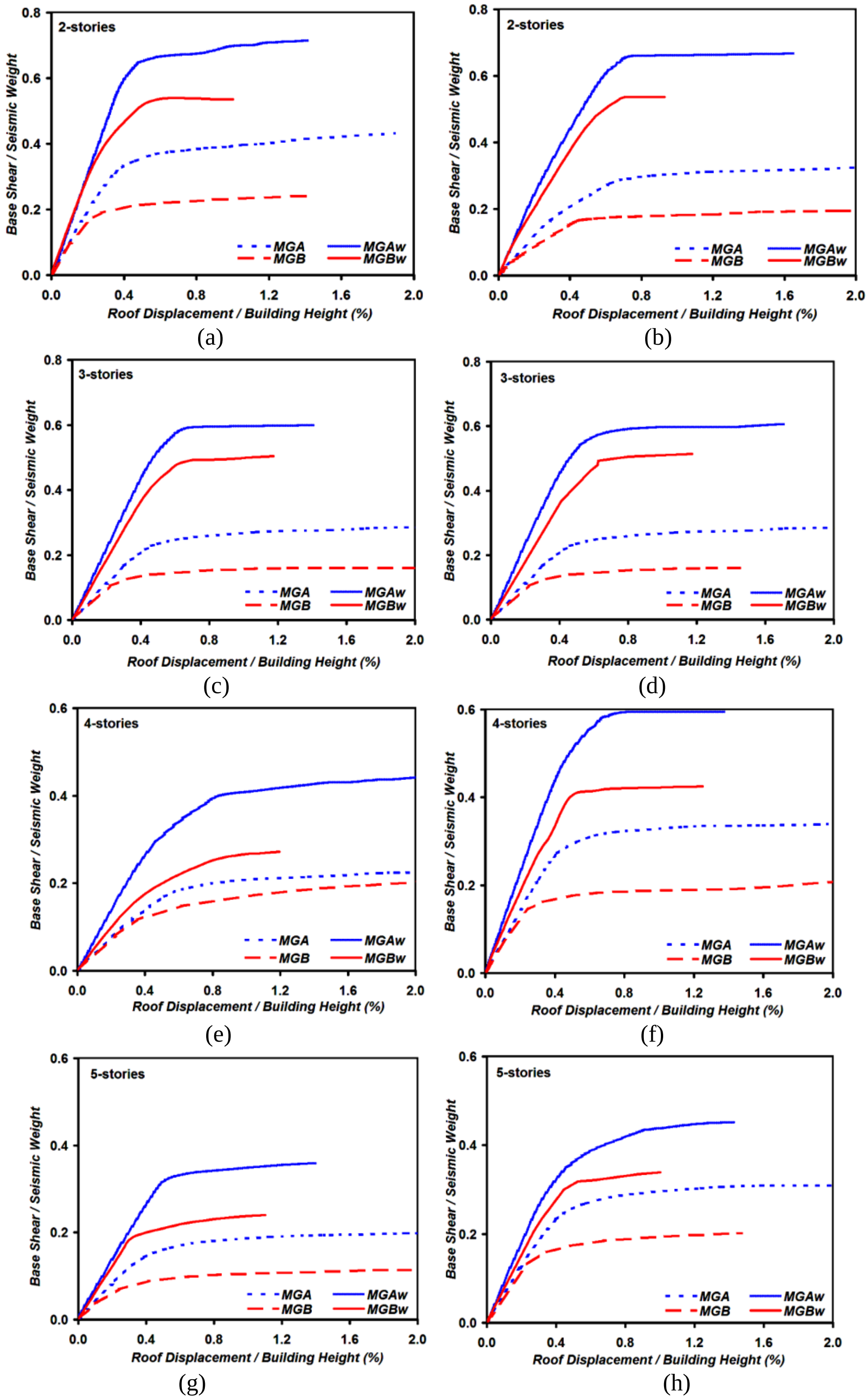


Fig. 3 Normalized pushover curves of selected some existing buildings

analyses, the locations of the masonry infill walls were determined according to the building architectural plan, and the infill walls were modelled as equivalent strut model. The thickness of infill wall was considered as constant (230 mm). The plan views of some selected buildings with infill walls are given in Fig. 2.

Numerous analyses were performed to investigate the effects of the masonry infill wall without opening on the seismic performance of buildings. For this reason, the three-dimensional models of each building were created by using SAP 2000 software program, and the performance analyses of the all buildings were carried out.

In the modeling of nonlinearity, three types of plastic hinges were considered which are flexural plastic hinges

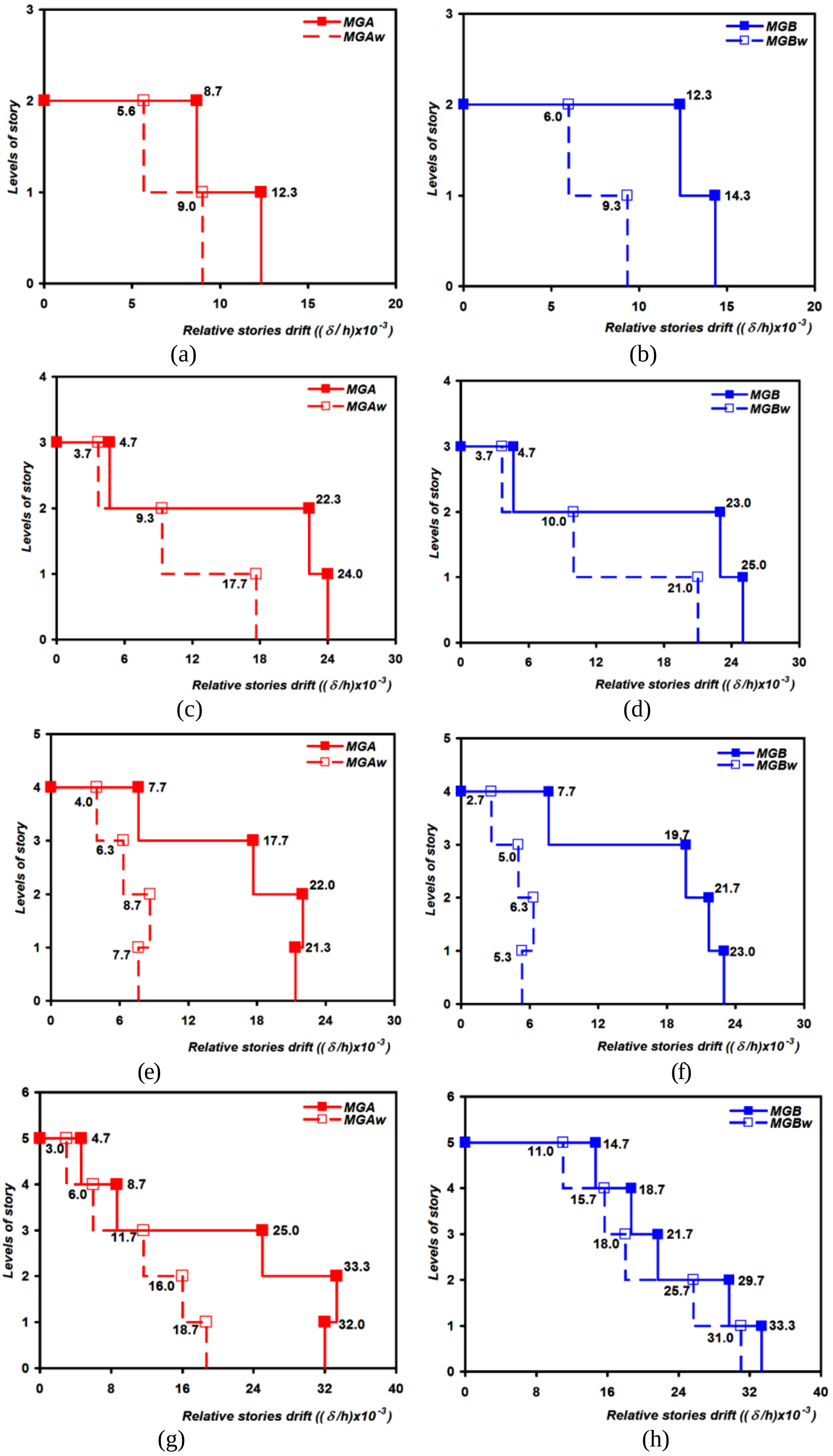


Fig. 4 The effect of masonry infill walls on relative stories drift

(M_2, M_3) for beams, compound compression and bending plastic hinges (PM_2M_3) for columns, and axial plastic hinges (P) for infill walls. In this study, the axial plastic hinge model proposed by Panagiotakos and Fardis (1996) was used for infill walls. Gravity and seismic loads were considered by assuming the design ground acceleration of 0.4 g (first seismic zone) and the soil class C according to FEMA 356. Seismic performances of the selected existing RC buildings were determined by considering two different cases that the first case is defined as “Material Group A”

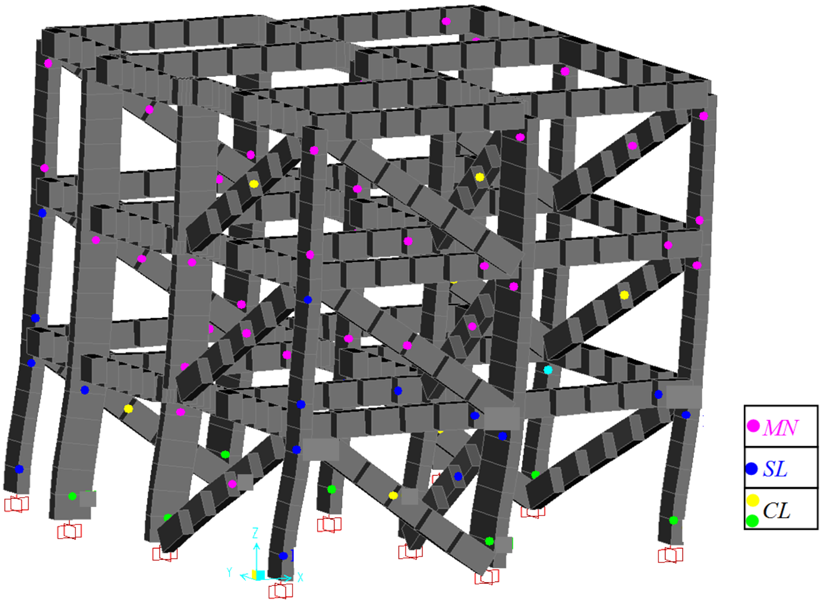


Fig. 5 The damage level on column, beam and diagonal struts

(MGA) and the second case was defined as "Material Group B" (MGB), respectively. In MGA, concrete strength was 20 MPa, steel yield strength was 420 MPa and spacing of transverse reinforcement was 100 mm. In MGB, concrete strength was 10 MPa, steel yield strength was 220 MPa and spacing of transverse reinforcement was 250 mm. The buildings with and without infill walls were analyzed for both cases, and the analysis results of buildings with and without infill wall were represented as MGA_w-MGB_w and *MGA-MGB*, respectively. Structural properties and analysis results of some selected existing buildings were given in Table 1. This study can be extended by considering different mechanical features of masonry infill walls with and without openings.

The pushover curves of some selected existing buildings with and without masonry infill walls were obtained from analysis as given in Fig. 3.

The relative story drifts were obtained for each floor level of the buildings with and without infill walls (MGA_w–MGB_w and *MG –MGB*) (Fig. 4).

The damage levels were checked according to strain level of concrete and steel in columns and beams. The seismic performance levels of buildings were determined by considering damage levels in column and beam elements in each story level. Analyzes have been performed for MGA_w–MGB_w and *MGA–MGB* cases.

In TEC, three damage limits are defined at the cross section level for ductile members. They are minimum damage limit (MN), safety limit (SL) and collapse limit (CL) of a cross-section. MN defines the onset of significant post-elastic behavior at a critical cross-section. Brittle members are not permitted to exceed this limit. A member damage state is determined by its critical cross-section with the most severe damage state. The critical sections are assumed to be at the ends of the columns and the beams. The seismic performance of a building is defined by using seismic damage levels of the structural elements. Corresponding performance regions and their limits can be

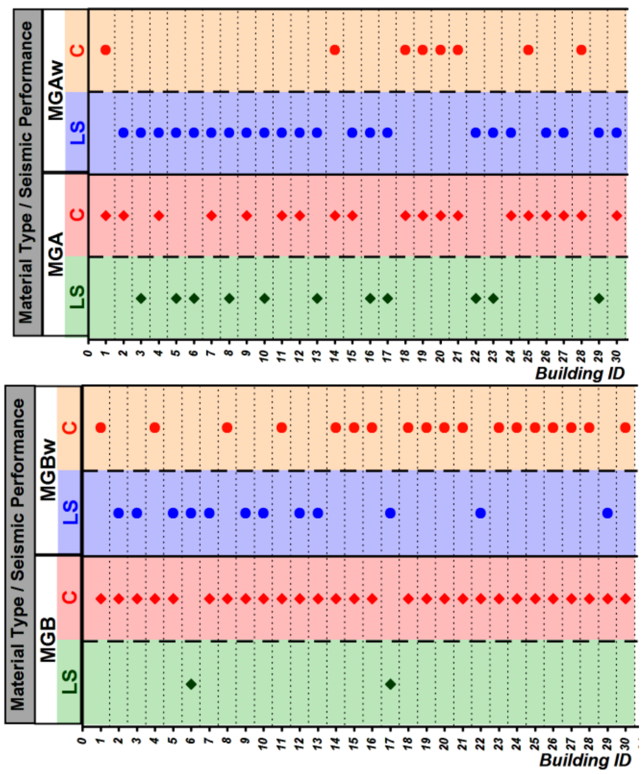


Fig. 6 The seismic performance of the two-storied buildings

defined for a building. As shown in Fig. 5, the initial damage was formed on the diagonal struts and damage continued on the columns and beams at the end of the pushover analysis.

The obtained results showed that the seismic performance level of the buildings is significantly affected by the parameters in material cases. Most of the buildings, which provided collapse performance level (C) in MGB case, are in life safety performance level (LS) in MGA case. The analyses results showed that damage levels in the columns are more affected than damage levels in beams in

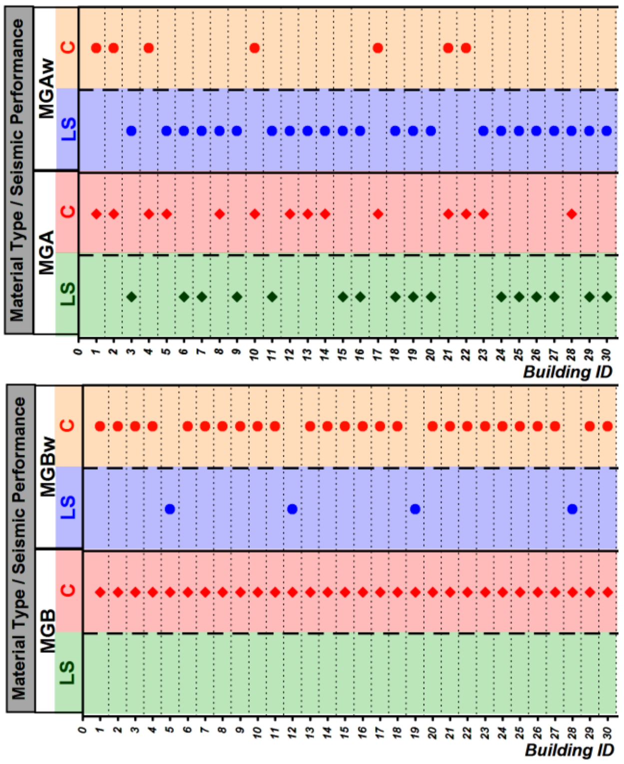


Fig. 7 The seismic performance of the three-storied buildings

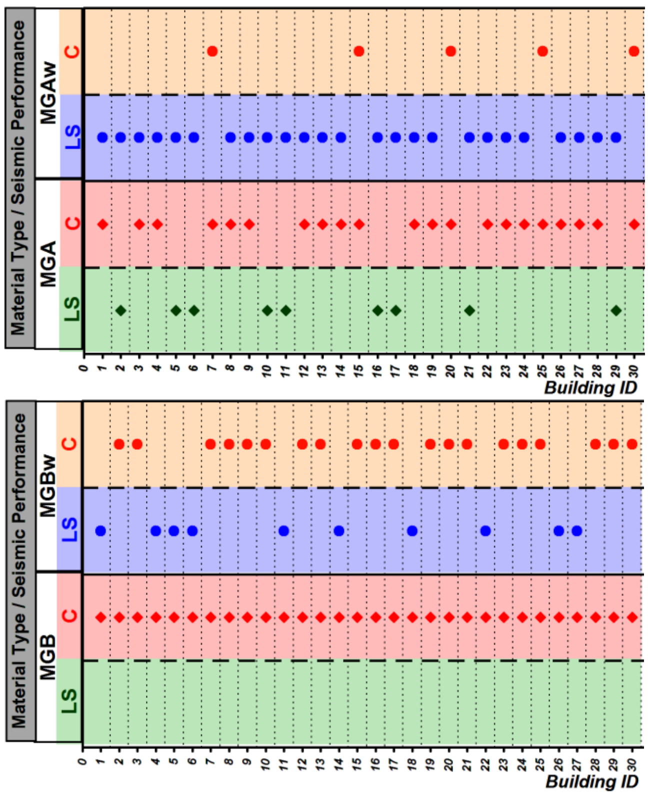


Fig. 8 The seismic performance of the four-storied buildings

determining the seismic performance of buildings. The performance level results of the selected existing buildings with and without infill walls are given comparatively in Figs. 6-9 according to number of stories, respectively.

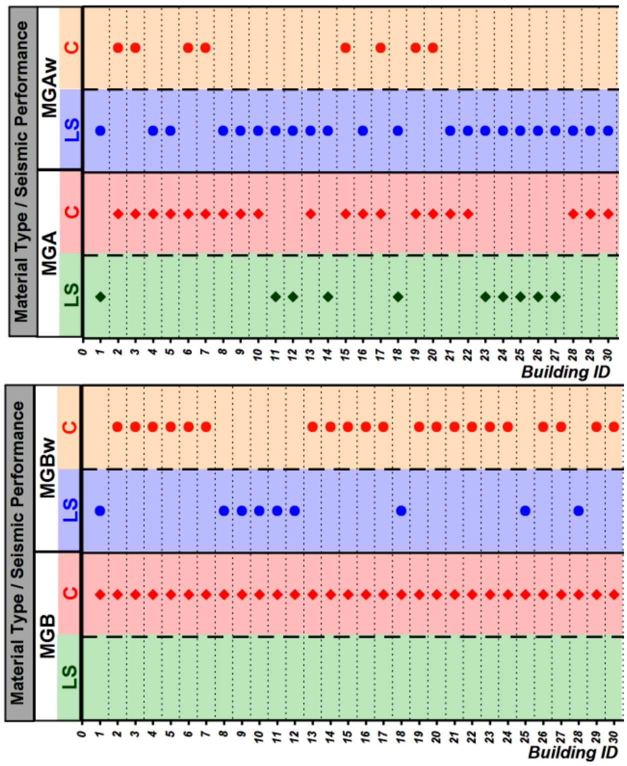


Fig. 9 The seismic performance of the five-storied buildings

Table 2 The percentages of the seismic performance levels

	MGA		MGAw		MGB		MGBw	
n	LS	C	LS	C	LS	C	LS	C
2	37%	63%	73%	27%	7%	93%	40%	60%
3	53%	47%	77%	23%	0%	100%	13%	87%
4	30%	70%	83%	17%	0%	100%	33%	67%
5	33%	67%	73%	27%	0%	100%	30%	70%

The effect of masonry infill walls on seismic performance of buildings can be seen in these figures. While the seismic performance levels of buildings without infill walls were generally obtained as “C” performance level, the performance levels of the buildings with infill wall were obtained as “LS” performance level. The performance levels of the existing buildings were given in Table 2 for the all material cases by comparing the results obtained from the analytical assessment. Additionally, the ratio percentages of the existing buildings determined according to the seismic performance levels were given also in the same table.

The percentages of the seismic performance levels of the existing building according to MGA, MGAw, MGB, MGBw and performance levels are as shown in Table 2

5. Conclusions

Many of the buildings are at the potential risk of loss of life and property in the earthquakes. It is known that influence of infill walls on the behavior of frames, which are subjected to earthquake loadings, is very essential in some cases. If the infill walls are uniformly distributed throughout the structure, then they usually have a beneficial effect on the seismic performance of the RC buildings. In this study, the seismic performances of RC frame buildings with and without masonry infill walls were evaluated by considering nonlinear behavior of reinforced concrete components. The existing RC buildings with 2, 3, 4 and 5 story and different infill wall configurations were selected. Infill walls were modeled by nonlinear strut elements, which only had compressive strength. The buildings with and without masonry infill walls were subjected to nonlinear analysis in order to assess seismic performance.

The analytical results of this current study indicated that masonry infill walls have very important effects on building seismic behavior, structural shear capacity and relative story displacement. The interaction between the bounding frame and the infill wall can lead to a considerable change in the distribution of the shear force in the columns of a story.

From obtained results, it was observed that the presence of infill wall is very effective on the lateral load-carrying capacity of RC buildings. In two, three, four and five storied RC buildings, the masonry infill walls can considerably increase the lateral load-carrying capacity of the buildings approximately five, four, three and two times, respectively. Results from indicated that the presence of infill walls in analysis of the buildings having MGA is more effective than in those of MGB on the pushover curves of the RC existing building.

Additionally, an increase in initial stiffness, strength, and energy dissipation of the infill walled buildings occurred with compared to the bare frame, despite the wall's brittle failure modes. Therefore, the buildings with infill walls has the lowest collapse risk for both MGA_w and MGB_w, and the bare framed buildings are found to be the most vulnerable to earthquake-induced collapse.

It had been observed that the cracked section period of the RC building changes depending on the effect and amount of the infill walls. The masonry infill walls caused a significant change in the building fundamental period between 15% and 50% as it directly affects building rigidity. Additionally, the effect of material classes was investigated on the fundamental period and this effect was observed to be between 5% and 10% for all material cases.

The relative story drifts obtained from analysis of the buildings having MGA material case were higher than those of MGA_w and MGB_w. In particular, the infill wall decreases the lateral displacements of stories of buildings by up to 50%-80% for all material cases.

Consequently, the infill walls affect the lateral load carrying capacity, the cracked section period, the relative stories drift and target seismic performance of the RC buildings to a large extent.

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