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Seismic response of RC frame structures strengthened by reinforced masonry infill panels

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Abstract. The performance of masonry infilled frames during the past earthquakes shows that the infill panels play a major role as earthquake-resistant elements. Experimental observations regarding the influence of infill panels on increasing stiffness and strength of reinforced concrete structures reveal that such panels can be used in order to strengthen reinforced concrete frames. The present study examines the influence of infill panels on seismic behavior of RC frame structures. For this purpose, several low- and mid-rise RC frames (two-, four-, seven-, and ten story) were numerically investigated. Reinforced masonry infill panels were then placed within the frames and the models were subjected to several nonlinear incremental static and dynamic analyses. In order to determine the acceptance criteria and modeling parameters for frames as well as reinforced masonry panels, the Iranian Guideline for Seismic Rehabilitation of Existing Masonry Buildings (Issue No. 376), the Iranian Guideline for Seismic Rehabilitation of Existing Structures (Issue No. 360) and FEMA Guidelines (FEMA 273 and 356) were used. The results of analyses showed that the use of reinforced masonry infill panels in RC frame structures can have beneficial effects on structural performance. It was confirmed that the use of masonry infill panels results in an increment in strength and stiffness of the framed buildings, followed by a reduction in displacement demand for the structural systems.

Keywords: seismic strengthening; reinforced masonry infill panels; nonlinear static analysis; nonlinear incremental dynamic analysis; low- and mid-rise reinforced concrete frames

1. Introduction

In recent years, a couple of studies have been conducted in the field of seismic rehabilitation and strengthening of buildings (Binici *et al*. 2007, Ozden *et al*. 2011, Akın and Ozcebe 2013, Akın *et al*. 2014). The results prove that reinforced masonry infill panels alter the response of reinforced concrete (RC) frame structures in terms of stiffness, strength and ductility (Dolsek and Fajfar 2008, Uva *et al*. 2012, Celarec *et al*. 2012, Ahmady *et al*. 2013, Cavaleri *et al*. 2014). Moreover, infill panels can cause significant changes in dynamic properties of the structure such as period, ductility, and seismic performance factor. The question of whether or not these changes are considered as beneficial is usually dependent on the distribution of infill panels in plan and

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elevation. A regular distribution of infill panels generally indicates, especially for non-seismic designed buildings, a beneficial effect, increasing global bearing capacity and stiffness under lateral actions. On the other hand, irregular distributions of panels may be dangerous, often being the cause of additional torsional effects, in the case of planar irregularities, and of soft-story mechanisms in the case of elevation irregularities (Cavaleri and Trapani 2014). Various methods have been developed for finite element modeling of infill panels, in order to determine the actual strength and stiffness of the structures. A particularly effective and widespread approach for representing the combined response of the frame and the masonry infill panels under the seismic actions is the use of equivalent diagonal struts (Saneinejad and Hobbs 1995). According to previous studies, the initial stiffness of frames can be calculated by using this methodology with reasonable accuracy.

Many researches have been done on the behavior of infill panels over the past five decades. Stafford Smith (1962) conducted several tests on steel frames filled with concrete panels. The results implied that the infill panel can be replaced by an equivalent diagonal strut. Furthermore, the force-displacement curve for the composite frames indicated that infill panels increase the stiffness of the infilled frames as compared to the unfilled ones. Saneinejad and Hobbs (1995) introduced a method for analysis and design of steel frames with concrete or masonry infill panels subjected to in-plane forces. This method was based on the data obtained from previous experiments as well as the results from different nonlinear finite element analyses. Since this method accounted for elastic and plastic behavior of infilled frames and considered the limited ductility of infilling materials, it had the capability to predict the strength and stiffness of infilled frames.

Mehrabi *et al.* (1996) investigated the influence of masonry infill panels on the performance of reinforced concrete frames. They tested twelve single-story, single-bay frame specimens subjected to monotonically increasing and cyclic loads. The results revealed that infill panels can dramatically improve the seismic performance of RC frames. Negro and Verzeletti (1996) conducted a series of pseudo-dynamic tests on a full-scale four-story reinforced concrete building. They evaluated the performance of the structure with different infill patterns by means of simplified SDOF techniques. The results showed that the differences in the behavior of structures with different distributions of infill panels can be captured using simplified SDOF methods and based on energy considerations.

Madan *et al.* (1997) proposed an analytical macro model based on an equivalent strut approach integrated with a smooth hysteretic model for considering the effect of masonry infill panels in nonlinear analysis of frame structures. The hysteresis model accounted for stiffness and strength degradations as well as pinching effects. Murty and Jain (2000) studied twelve single-bay single-story frames experimentally, in order to probe the influence of infill wall panels on RC frame structures. The results showed that masonry infill panels increase strength, stiffness, overall ductility and energy dissipation of the building.

Santhi *et al.* (2005) studied two models of one-bay three-story space frames, one without infill and the other with a brick masonry infill in the first and second floors. It was observed that severe damage occurred in the columns of the ground floor. The damaged columns were then strengthened by a reinforced concrete jacket and tested under the same earthquake excitations. The results showed that the retrofitted frames could sustain low to moderate seismic forces due to a dramatic increase in strength and stiffness. Tasnimi and Mohebkhah (2005) studied the effect of mass and stiffness irregularities due to the presence of infill panels on reinforced concrete frame structures. They adopted the equivalent diagonal strut method in order to include the infill panel stiffness in the frame models. It was found that the infill panels considerably decrease the story drift demands and increase the strength of the whole structure.

Fiore *et al.* (2012) performed numerical analyses on two models of RC existing structures, both for the bare frames and for the infilled frames, in order to evaluate the variation of structural capacity due to the interaction of infill panels with RC elements. It was confirmed that the presence of the infill panels results in an increase in the stiffness and the strength of the overall structure, accompanied by a reduction in the displacement capacity.

Mondal and Tesfamariam (2014) quantified the effects of vertical irregularity and thickness of unreinforced masonry infills on the robustness of a six-story three-bay RC frame structure. It was observed that the infill thickness and vertical irregularity have significant influence on the response of the RC frame structure. Al-Nimry *et al.* (2014) conducted a study with an aim to investigate the fundamental vibration period of infilled RC frame buildings using measurements of ambient vibrations and numerical analyses. It was found that the periods of vibration obtained for RC frame buildings with infill panels were much shorter than the values estimated by international building codes.

The present study examines the impact of infill panels on seismic response of RC frame structures. For this purpose, several low- and mid-rise RC frames (two-, four-, seven-, and ten-story) were modeled in SAP2000 software. Reinforced masonry infill panels were then placed within the frames and the models were subjected to several nonlinear incremental static and dynamic analyses. It should be noted that the guidelines adopted in this research so as to define the acceptance criteria as well as modeling parameters for frames and reinforced masonry panels include the Iranian Guideline for Seismic Rehabilitation of Existing Masonry Buildings (Issue No. 376), the Iranian Guideline for Seismic Rehabilitation of Existing Structures (Issue No. 360), and FEMA Guidelines (FEMA 273 and 356).

2. Description of models

Models studied in this research consist of four two-dimensional reinforced concrete frames

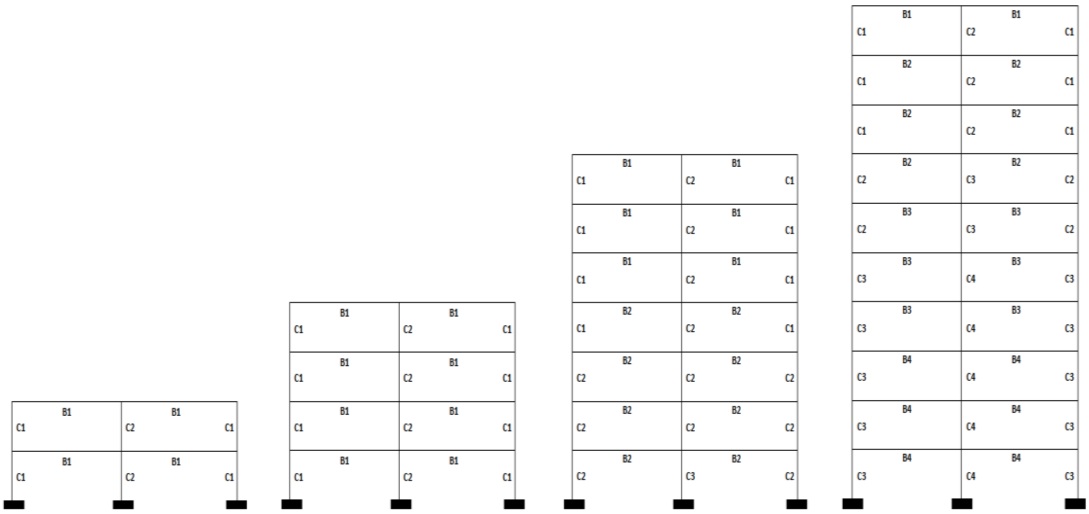


Fig. 1 Schematic view of the frame structures

Table 1 Dimensions of beams and columns used in the frames

Section name	Dimensions (cm)
Two-story frame	
C1	40×40
C2	45×45
B1	40×40
Four-story frame	
C1	40×40
C2	45×45
B1	40×40
Seven-story frame	
C1	40×40
C2	45×45
C3	50×50
B1	40×40
B2	45×45
Ten-story frame	
C1	45×45
C2	50×50
C3	55×55
C4	60×60
B1	40×40
B2	45×45
B3	50×50
B4	55×55

with two, four, seven, and ten stories, and with the average story height of 3.2 meters. Gravity and seismic loads were assigned to the frames according to the criteria in the 6th section of the Iranian National Building Code (INBC-06 2006) and the Iranian Code of Practice for Seismic Resistant Design of Buildings (BHRC 2005). Moreover, the buildings were considered as ordinary buildings with residential occupancy, and are supposed to be built in a site with conditions matching ground type *II*. The construction site is also located in a region of high seismicity (Design Base Acceleration Ratio=0.35). The specifications of the beams and columns used in the structures under study are appeared in Fig. 1 and Table 1.

3. Modeling of reinforced masonry infills

Vertical and horizontal bars can be used inside the reinforced masonry infill panels in order to prevent flexural and shear failure modes. Combination of steel and masonry materials in this way

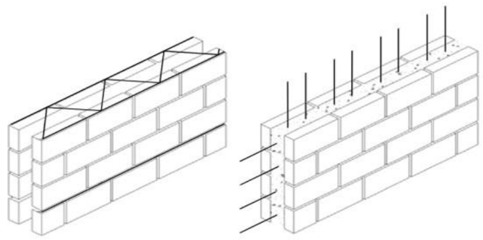


Fig. 2 Sample models of efficient reinforcement for a brick wall

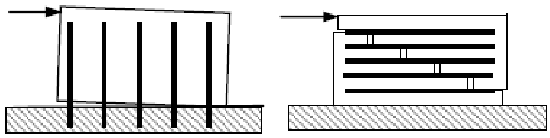


Fig. 3 Vertical and horizontal bars used in walls in order to prevent flexural and shear failure

produces a material with properties rather similar to reinforced concrete. It should be noted that, however, the horizontal bars are placed into the wall while their diameter cannot actually exceed 10 or 12 mm. In addition, the vertical bars must be hooked into the lower horizontal strap in the wall so as to transfer the bending moments due to earthquakes into the foundation. In Fig. 2, two models of efficient reinforcement for a brick wall have been illustrated (Derakhshan *et al.* 2009). It is also noteworthy that since the infill panels are confined by the frames, the panels are mainly exposed to shear forces rather than bending moments (Fig. 3). This is why it is necessary to use sufficient horizontal bars in the design of reinforced masonry infill panels.

3.1 Equivalent compressive strut

As shown in Fig. 4, divergent and convergent diagonal strut modeling can be adopted in order to include the effect of infill panels' stiffness in the structural model. The divergent strut modeling method assumes that the infill panels are only in touch with the adjacent columns. Therefore, the interactions due to lateral deformations would be limited only to those between the infill panel and the adjacent columns. This is while the convergent strut modeling assumes that the infill panel is in touch not only with the adjacent columns, but also with the adjacent beams. In both cases, the width of the diagonal strut is calculated from the Eq. (1) (SRES-360 2006)

$$W = 0.254[\lambda h]^{-0.4} d, \lambda = \left[10 E_i t \sin 2\theta / E_f I_{col} h\right] \tag{1}$$

in the above equation:

h is the height of the infill panel (cm),
d is the length of the equivalent strut (cm),
E_f is the modulus of elasticity for the frame materials (kg/cm^2),
E_i is the modulus of elasticity for the infill panel materials (kg/cm^2),
I_{col} is the moment of inertia for the column (cm^4),
t is the thickness of the infill panel and the equivalent diagonal strut (cm),
θ is an angle with a tangent equal to the ratio of the height to the length of infill panel's span, and
w is the effective width of the equivalent strut.

Among the aforementioned models, the convergent model outperforms the divergent one, because of the fact that bending and shear forces in infilled frame members are not well displayed in the divergent modeling procedure. Meanwhile, the convergent model is capable of better

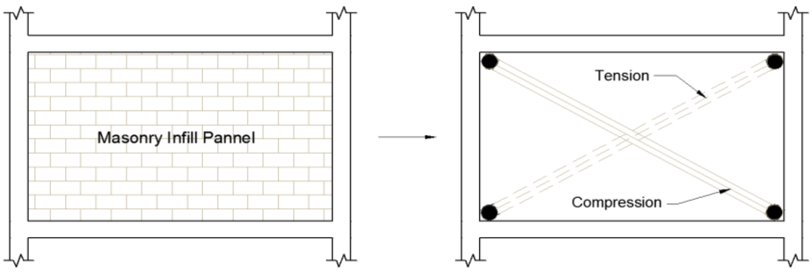


Fig. 4 Indication of the equivalent compressive and tensile struts in the model as diagonal pairs

displaying the actual stressed area of the infill panels. In addition, since the contact length between the frame and the infill panel is the most important issue in transferring forces from the frame members to the infill panels, the convergent model allows considering different types of infill panels interaction in multi-story buildings (El-Dakhakhni *et al.* 2003). Accordingly, the convergent diagonal strut was implemented in models of this study.

3.2 Calculation of the effective width for the equivalent strut

According to the Iranian Guideline for Seismic Rehabilitation of Existing Structures (SRES-360, 2006), an equivalent compressive strut should be used in order to simulate the panel behavior of masonry infill material. Moreover, the modulus of elasticity and thicknesses for the equivalent strut and the infill panel should be identical. As shown in Fig. 5, the in-plane stiffness of the uncracked masonry infill panel can be estimated through Eq. (1), by applying an equivalent compressive diagonal strut. In current study, the effective width of equivalent struts was calculated considering the changes in the infill panel material strength, bay length, and stiffness of columns adjacent to the infill panels. It should be noted that the strut thickness is considered to be equal to that of the infill panel, i.e., 20 cm. The effective width of equivalent struts used in the frames is shown in Table 2.

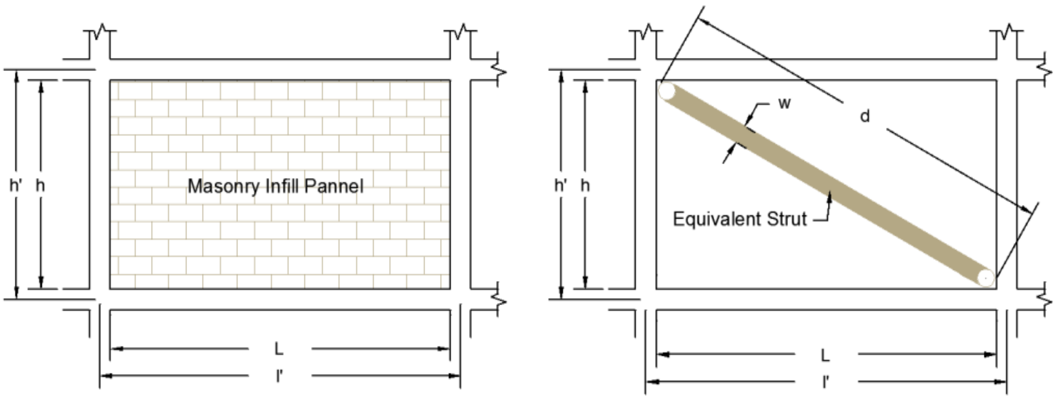


Fig. 5 Modeling infills through single compressed strut

Table 2 Effective width of equivalent struts used in the frames

	Story	E_f (kg/cm^2)	E_i (kg/cm^2)	λ_1	w (cm)
Two-story frame	1	299000	39600	0.0223	61.73
	2	299000	39600	0.0223	61.73
Four-story frame	1	299000	39600	0.0223	61.46
	2	299000	39600	0.0223	61.46
	3	299000	39600	0.0223	61.46
	4	299000	39600	0.0223	61.46
Seven-story frame	1	299000	39600	0.0201	63.56
	2	299000	39600	0.0201	63.56
	3	299000	39600	0.0223	61.46
	4	299000	39600	0.0223	61.46
	5	299000	39600	0.0223	61.82
	6	299000	39600	0.0223	61.82
	7	299000	39600	0.0223	61.82
Ten-story frame	1	299000	39600	0.01883	64.07
	2	299000	39600	0.01883	64.07
	3	299000	39600	0.01883	64.07
	4	299000	39600	0.01883	64.45
	5	299000	39600	0.01883	64.45
	6	299000	39600	0.01883	64.45
	7	299000	39600	0.02060	62.95
	8	299000	39600	0.02060	62.95
	9	299000	39600	0.02060	62.87
	10	299000	39600	0.02060	62.87

3.3 Definition of infill panels' material in numerical models

Various methods have been proposed to simulate the nonlinear behavior of material in numerical models. All of these procedures are applied based on the similarity to the actual behavior of material as well as the availability of information. Because of the fact that compressive, tensile and shear standard tests can be easily done for masonry and concrete material, it is perfectly logical to use nonlinear behavior of such material in finite element models. Accordingly, the specifications of members and components used in the models were extracted in accordance with the criteria contained in the adopted guidelines.

3.4 Modeling parameters and acceptance criteria for reinforced masonry walls

As it was mentioned before, since the infill panels are confined with frames, the panels are mainly exposed to the shear force rather than bending moment. Consequently, axial and shear hinges were employed in order to define plastic hinges in diagonal strut elements. Moreover, the transfer of shear forces along a masonry panel confined in a concrete or steel frame, and the transfer of axial forces in diagonal members are both controlled by deformation. Therefore, the Table 7-7 in FEMA 356 guideline was used in order to define plastic hinges in reinforced infill panels.

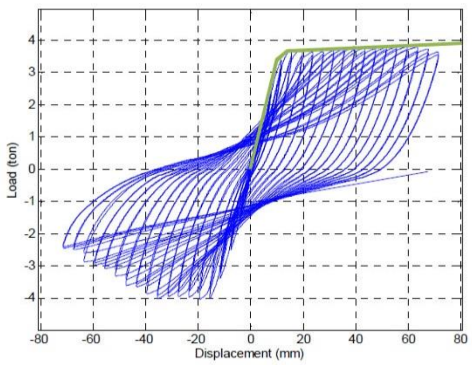
It is also noteworthy that since the infill panels cross each other in the frame, the hinges were assumed to be located at distances (0.05*L*) and (0.95*L*) along the members, where *L* is the length of the corresponding member. Apparently, in the case where the diagonal element members are separately used, the best location for the hinge is the middle part of the diagonal element.

4. Verification of models

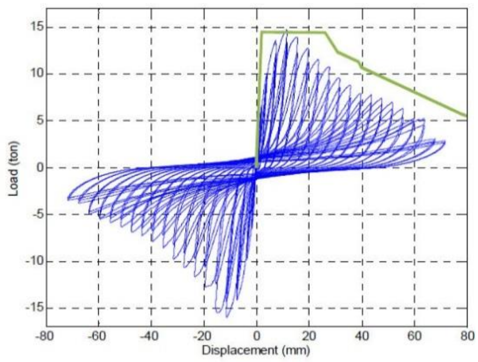
The results of a research conducted in International Institute of Earthquake Engineering and Seismology (Parsa and Moghadam 2006) were used in order to validate the numerical modeling performed in current study. For this purpose, a model similar to that presented in aforementioned research was made in SAP2000 software. The model was then calibrated based on the data obtained from the research. A nonlinear static analysis was performed after loading and defining the hinges in beams, columns and infill panels. Afterwards, the results were compared with each other and it was investigated that the results are in good agreement. As shown in Fig. 6, the results of the nonlinear static analysis for the structures without infill panels are in apparent consensus with the curve obtained from cyclic loading. This actually reflects the validity of the finite element model. Moreover, comparison of the results for the structures with infill panels reveals that the analytical model has a higher strength than the experimental model used in abovementioned research. The reasons for this can be explained as follows:

• In considered research, unreinforced masonry infill panels are used, whereas in the present study, reinforced masonry infill panels are employed.

• The loads applied to the structure in aforementioned research were cyclic, while the structural loads are monotonic in the present study. Obviously, the strength will further decrease during the reciprocating cycles compared to the monotonic loading.



Results for bare frames



Results for infilled frames

Fig. 6 Comparing the results of nonlinear static analysis with experimental results for the single-story single-bay bare and infilled frames

5. Methods of analysis

In this study, several nonlinear static and dynamic analyses have been employed.

5.1 Nonlinear static analysis

The nonlinear static procedure, colloquially known as pushover analysis, has become a standard method for estimating seismic deformation demands in building structures as well as their local and global capacities (Nazri and Ku 2014). Pushover analysis provides a lot of information on many response quantities which cannot be obtained from a linear static or dynamic analysis. Estimation of the inter-story drift and its distribution over the height of the building can be mentioned as a major benefit of pushover analysis. In this method, the amount of lateral loads increases gradually until the drift of the roof of the building reaches a significant level or the building loses its stability. Since the lateral load distribution should be similar to what happens during a real earthquake, it is usually recommended to use at least two types of load distribution pattern while performing such analysis. The lateral load patterns used for pushover analysis in current study are briefly described below.

5.1.1 Uniform lateral force distribution pattern (Accel)

In this pattern, the force acting on each story is calculated in proportion to its weight (Eq. (2)) (MPOIRI 2006)

$$\Delta F_i = \frac{W_i}{\sum_{j=1}^{N} W_j} \Delta V \tag{2}$$

where N is the number of stories above the foundation.

5.1.2 Equivalent lateral force distribution pattern (Push)

This pattern is the same as the lateral load distribution pattern proposed in different building codes for the equivalent static analysis (Eq. (3)) (MPOIRI 2006, BHRC 2005). It is used when at least 75% of the total mass of the structure is involved in the first mode of vibration.

$$\Delta F_i = \frac{W_i h_i^k}{\sum_{j=1}^{N} W_j h_j^k} \Delta V \tag{3}$$

The proposed value for k is calculated as a function of the fundamental period of the structure (T)

$$\begin{cases} k = 1 & T \le 0.5 \\ k = 1 + \dfrac{T - 0.5}{2} & 0.5 \prec T \prec 2.5 \\ k = 2 & T \ge 2.5 \end{cases}$$

5.1.3 Lateral force distribution according to the first mode shape (Mode 1)

This distribution pattern is appropriate to the shape of the first mode of vibration, (Eq. (4)). It is used when at least 75% of the entire mass of the structure is involved in the first mode (MPOIRI 2006)

$$\Delta F_i = \frac{W_i \Phi_{i1}}{\sum_{j=1}^{N} W_j \Phi_{j1}} \Delta V \tag{4}$$

where Φ_{i1} stands for the relative displacement of the *i*-th story in the first mode shape. It is also noteworthy that *F*, *W*, *h*, and *V* in the above equations stand for story force, story weight, story height, and design base shear, respectively.

5.2 Incremental dynamic analysis

The incremental dynamic analysis (IDA) is a parametric analysis which has recently emerged in various forms for evaluating the overall performance of structures under seismic loads. This concept was first proposed by Bertero in 1977. Various researchers studied the concept and finally turned it into a general method. This procedure contains subjecting a structural model to several ground motion excitations, each scaled to various levels of intensity. Accordingly, numerous curves of response are produced that are all parameterized versus intensity level. In this study, the incremental dynamic analysis was used as a complete practical method to identify the overall capacity curve as well as the yield mechanisms of structural members under the selected earthquakes.

The IDA study is now a multi-purpose and broadly applicable method and some of its objectives include (Vamvatsikos and Cornell 2002):

• superior understanding of the range of response or demands versus the range of potential levels of a ground motion record,

• thorough understanding of the structural implications of more or less severe ground motion levels,

• better understanding of the alterations in the nature of the structural response as the ground motion intensifies, and

• providing estimates of the dynamic capacity of the overall structural system

Table 3 General specifications of Earthquakes

Region	Year	PGA (g)	PGV (cm/s)	PGD (cm)	Magnitude			ED* (km)	Database
					M	Ml	Ms		
San Fernando	1971	0.366	17	1.65	6.6	-	6.6	20.04	CDMG
Kobe	1995	0.821	81.3	17.68	6.9	-	-	18.27	-
Landers	1992	0.171	20.2	13.87	7.3	-	7.4	27.33	CDMG
Mexico	1980	0.621	31.6	13.2	-	6.1	6.4	33.73	UNAM
Northridge	1994	0.883	41.7	15.09	6.7	6.6	6.7	22.45	CDMG
N-Palm	1986	0.694	33.8	3.88	6.0	5.9	5.0	10.57	USGS
Tabas	1978	0.852	121.4	94.58	7.4	7.7	7.4	55.24	-

*ED: Epicentral Distance

The nonlinear response of buildings is highly sensitive to the modeling parameters and ground motion characteristics. Therefore, a single-record IDA cannot fully represent the behavior of a building under the impact of possible future earthquakes. In other words, a set of various ground motion records should be used in order to cover the whole range of responses. The results of the incremental dynamic analysis indicate that this method can be turned into a potentially valuable tool in earthquake engineering.

In the present study, seven ground motion records were used to perform nonlinear time history analyses, and individual capacity curves were obtained from each IDA. General Specifications of Earthquakes including the magnitude and Peak Ground Acceleration of selected records are also shown in Table 3. Moreover, the acceleration time histories of applied ground motions as well as their corresponding response spectrum with 5% damping ratio are depicted in Fig. 7.

6. Results of analyses

In current study, several 2D frames with and without infill panels were designed and analyzed through SAP2000 software. The side frames were regarded to be entirely infilled by reinforced masonry panels, considering the fact that the middle frames are rarely completely filled by infilling walls. Comparison of results obtained from incremental dynamic analyses has been shown in Fig. 8. Furthermore, the maximum PGA values in which the frames fail to maintain stability are

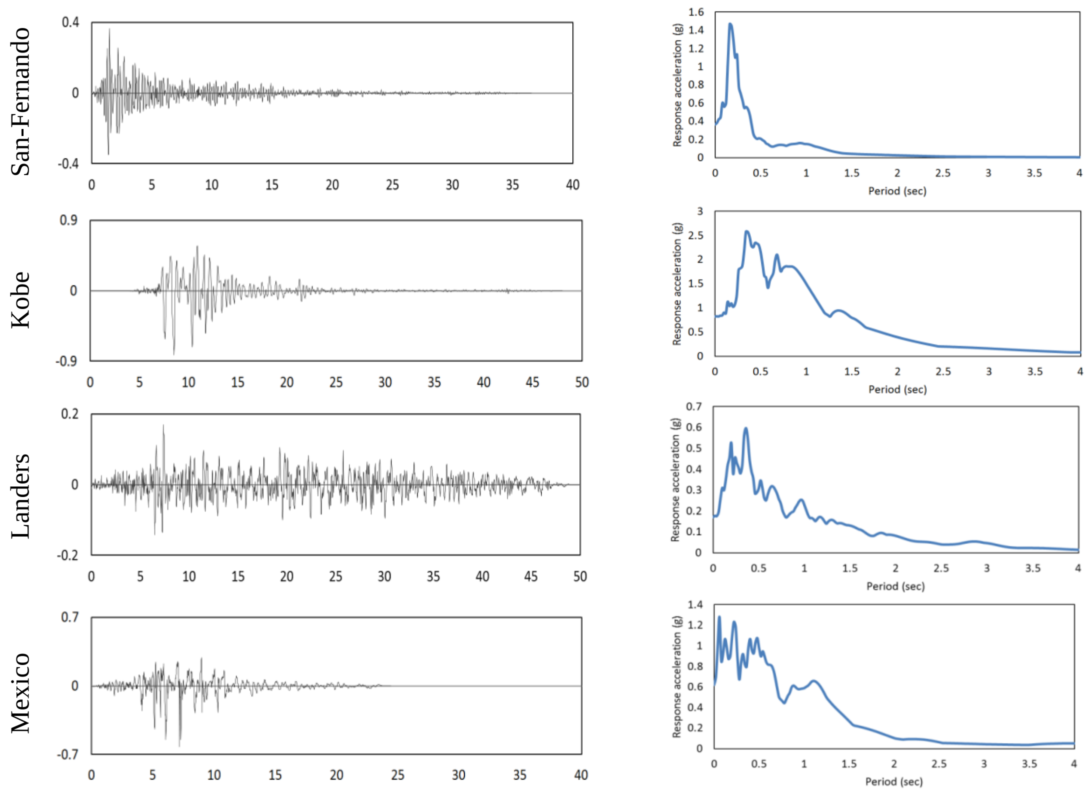
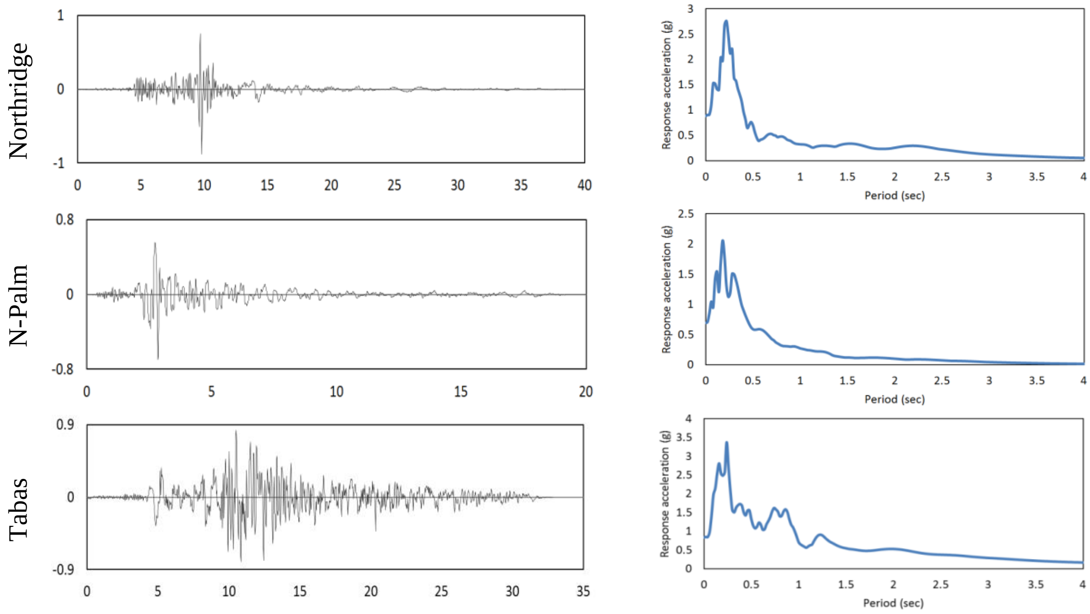
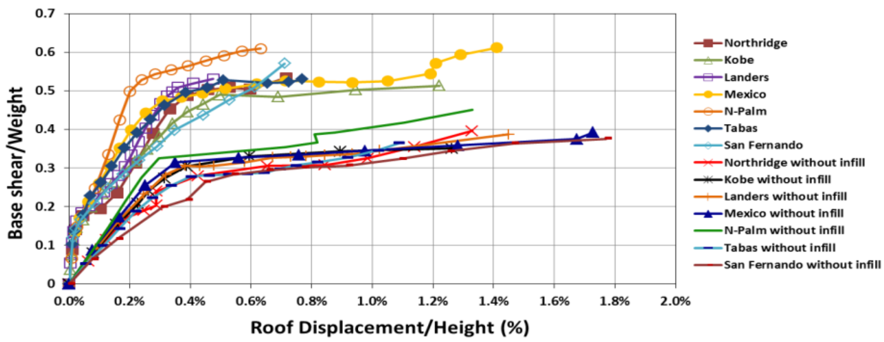


Fig. 7 Acceleration time history and normalized acceleration response spectrum of earthquakes

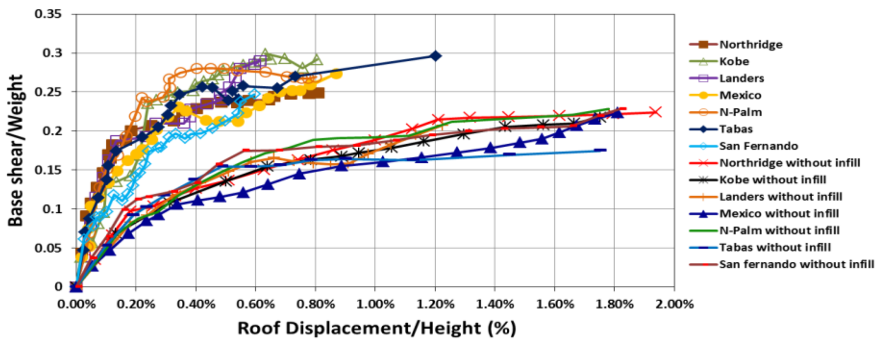


Acceleration time history Acceleration response spectrum

Fig. 7 Continued

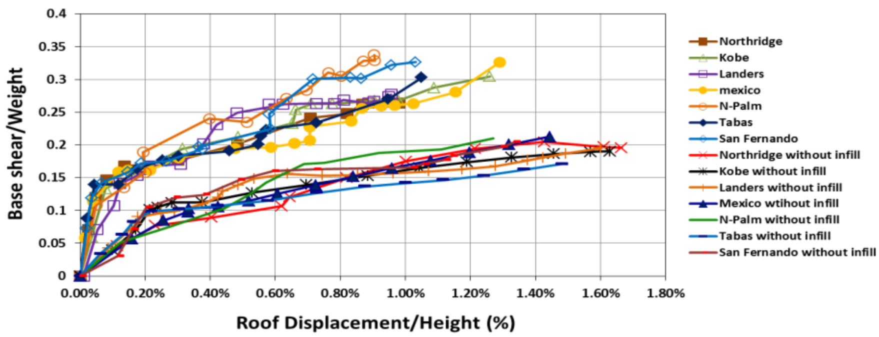


Two-story frames

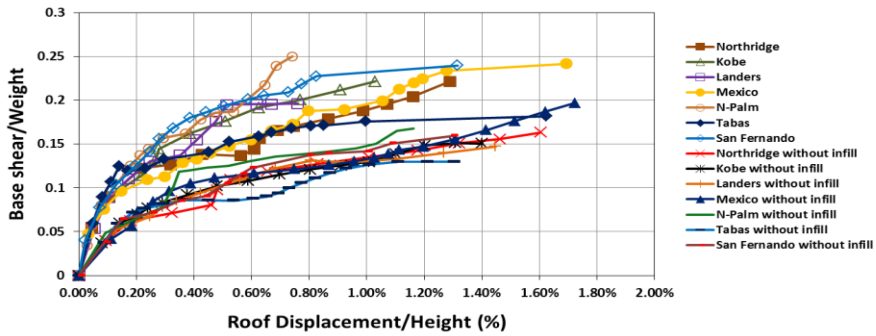


Four-story frames

Fig. 8 Comparison of the capacity curve for the bare and infilled frames

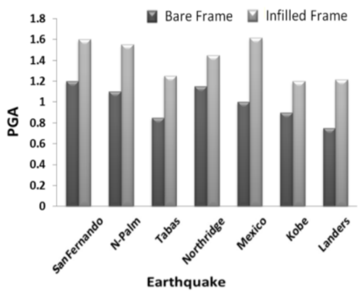


Seven-story frames

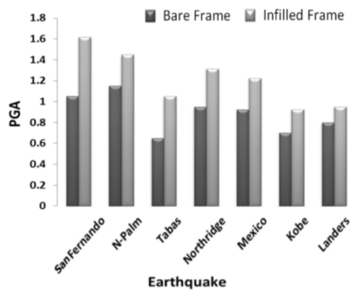


Ten-story frames

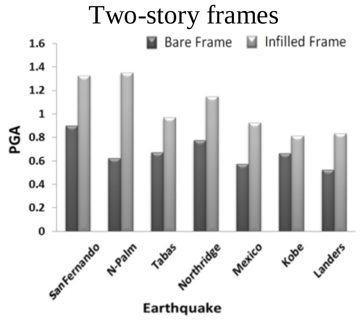
Fig. 8 Continued



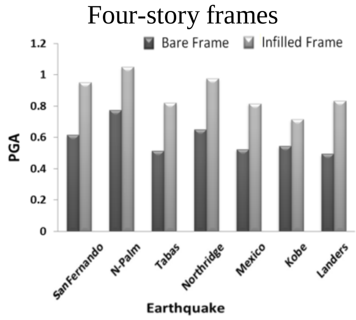
Two-story frames



Four-story frames



Seven-story frames



Ten-story frames

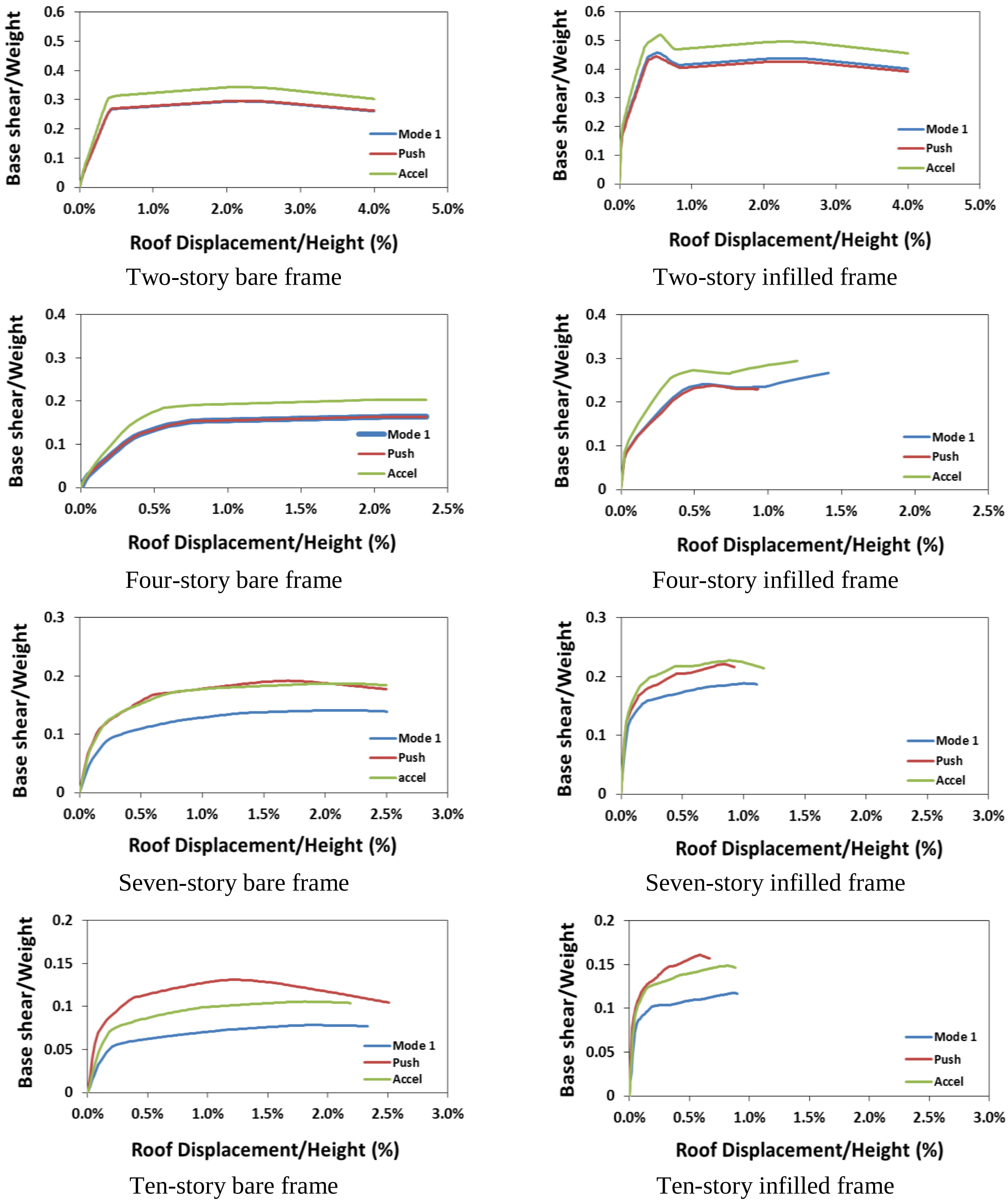
Fig. 9 Comparison of the maximum PGA values for the bare and infilled frames

Table 4 Comparison of the maximum PGA values for the bare and infilled frames

Two-Story Frame	Peak Ground Acceleration (g)						
	Landers	Kobe	Mexico	Northridge	Tabas	N-palm	San Fernando
Bare Frame	0.750	0.900	1.000	1.150	0.850	1.100	1.200
Infilled Frame	1.215	1.200	1.615	1.450	1.250	1.550	1.600
PGA Ratio	1.620	1.333	1.615	1.261	1.471	1.409	1.333
Average PGA Ratio				1.435			
Standard Deviation				0.131			
Coefficient of Variation				0.091			
Four-Story Frame	Peak Ground Acceleration (g)						
	Landers	Kobe	Mexico	Northridge	Tabas	N-palm	San Fernando
Bare Frame	0.800	0.750	0.925	0.950	0.650	1.150	1.050
Infilled Frame	0.925	0.925	1.225	1.315	1.050	1.450	1.620
PGA Ratio	1.156	1.233	1.324	1.384	1.615	1.261	1.543
Average PGA Ratio				1.359			
Standard Deviation				0.155			
Coefficient of Variation				0.114			
Seven-Story Frame	Peak Ground Acceleration (g)						
	Landers	Kobe	Mexico	Northridge	Tabas	N-palm	San Fernando
Bare Frame	0.525	0.665	0.575	0.775	0.675	0.625	0.900
Infilled Frame	0.835	0.815	0.925	1.150	0.970	1.350	1.325
PGA Ratio	1.590	1.226	1.609	1.484	1.437	2.160	1.472
Average PGA Ratio				1.568			
Standard Deviation				0.268			
Coefficient of Variation				0.170			
Ten-Story Frame	Peak Ground Acceleration (g)						
	Landers	Kobe	Mexico	Northridge	Tabas	N-palm	San Fernando
Bare Frame	0.495	0.545	0.525	0.650	0.515	0.715	0.615
Infilled Frame	0.640	0.715	0.815	0.975	0.820	1.050	0.950
PGA Ratio	1.293	1.312	1.552	1.500	1.592	1.469	1.545
Average PGA Ratio				1.466			
Standard Deviation				0.110			
Coefficient of Variation				0.075			

presented in Fig. 9 for each accelerogram. Besides, Table 4 demonstrates the average PGA ratios, the standard deviation, and the coefficient of variation of PGA values for each frame. As it is obvious, there is a good correlation between the PGA values. Finally, the results of nonlinear static analyses have been illustrated in Fig. 10. The capacity ratios as well as relative displacement ratios of the infilled frames to corresponding bare frames have also been presented in Table 5.

It should be noted that additional analyses have been performed to prove that although infill walls are regarded as nonstructural components in structural design, they can substantially contribute to improve the resistance of frame structures during earthquakes.



Two-story bare frame | Two-story infilled frame

Four-story bare frame | Four-story infilled frame

Seven-story bare frame | Seven-story infilled frame

Ten-story bare frame | Ten-story infilled frame

Fig. 10 Comparison of the pushover capacity curve for the bare and infilled frames

Table 5 Capacity and displacement ratios of the infilled frames to corresponding bare frames

incremental dynamic analysis:	capacity ratio		relative displacement ratio
	linear range	nonlinear range	
two-story frames	1.70	1.40	0.60
four-story frames	1.50	1.30	0.50
seven-story frames	1.70	1.25	0.65
ten-story frames	1.50	1.20	0.80
nonlinear static analyses:	capacity ratio		relative displacement ratio
	linear range	nonlinear range	
two-story frames	1.60	1.40	0.90
four-story frames	1.40	1.30	0.50
seven-story frames	1.50	1.35	0.45
ten-story frames	1.50	1.30	0.30

7. Conclusions

In this study, finite element models of four low- to mid-rise RC fame structures with and without reinforced masonry infill panels were made. A compression strut model for masonry panels was employed in order to describe the behavior of the infill panels. In brief, it is concluded that using reinforced masonry infill panels in RC frame structures can have beneficial effects on structural performance, and considerably changes the nonlinear behavior of the structure. It is confirmed that use of masonry infill panels results in an increment in strength and stiffness of the framed buildings, followed by a reduction in displacement demand for the structural systems. It is also worth mentioning that infill panels have a more positive influence on strength and stiffness of the structures in two, four, and seven-story frames compared to the ten-story frame. This shows that the use of infill panels in low-rise RC frame structures is an effective way of improving structural performance during earthquakes, because of the fact that stiffness is a crucially important characteristic of low-rise earthquake resistant buildings.

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