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Seismic assessment of R/C residential buildings with infill walls in Turkey

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Abstract. In 1999 Marmara and 2011 Van earthquakes in Turkey, majority of the existing buildings either sustained severe damage or collapsed. These buildings include masonry infill walls in both the interior and exterior R/C frames. The material of the masonry infill is the main variant, ranging from natural stones to bricks and blocks. It is demanding to design these buildings for satisfactory structural behavior. In general, masonry infill walls are considered by its weights not by interaction between walls and frames. In this study, R/C buildings with infill walls are considered in terms of structural behavior. Therefore, 5 and 8-story R/C buildings are regarded as the representative models in the analyses. The R/C representative buildings, both with and without infill walls were analyzed to determine the effects of structural behavior change. The differences in earthquake behavior of these representative buildings were investigated to determine the effects of infill walls leading structural capacity. First, pushover curves of the representative buildings were sketched. Aftermath, time history analyses were carried out to define the displacement demands. Finally, fragility analyses were performed. Throughout the fragility analyses, probabilistic seismic assessment for R/C building structures both with and without infill walls were provided. In this study, besides the deterministic assessment methodology, a probabilistic approach was followed to define structural effect of infill walls under seismic loads.

Keywords: structural irregularities in R/C structures; structures with infill walls; nonlinear pushover analysis; fragility analysis

1. Introduction

Turkey is under high seismic risk since it is located in one of the most active seismic regions of the world. Severe earthquakes, which caused enormous casualties, occurred in the last several decades in Turkey and it is probable that such devastating earthquakes and losses will take place again in the future. In recent Turkish earthquakes, many of the existing reinforced concrete structures did not perform well and suffered heavy damage or collapsed due to inadequate seismic behavior. In the most of the cases, poor performances have been attributed to the structures lack of ductility required to sustain the inelastic deformation demand (Irtem *et al*. 2007). Earthquakes

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cause also economic damages besides the life loss and building collapse.

Determination of earthquake performance of existing buildings is very essential to decrease the structural damage at the earthquakes. The need to predict the seismic vulnerability of existing buildings has led to increased interest on research dealing with the development of seismic vulnerability assessment techniques (Yakut 2004). Determination and assessment of structural damage from earthquake provide useful information for post-earthquake planning and risk mitigation studies. Also, information on structural damage is of critical importance for reliable economic loss evaluation for a structure or a region that has been or that might be affected by an earthquake (Singhal and Kiremidjian, 1996, Kamanli and Balik 2010).

Fragility curves and damage probability matrices are the two most common tools that are used to characterize the relationship between an earthquake intensity and structural damage. Fragility curves, which are also employed for determination of earthquake intensity-damage relation in this study, are the measure to determine the probabilistic earthquake damage levels. Probabilistic loss and damage assessment is based on the fragility analysis and they allow estimating the cumulative probability of structural damage of reaching or exceeding certain damage states as a function of various ground motion parameters (Arslan and Korkmaz 2007).

Existing reinforced concrete residential buildings have mostly structural irregularities in Turkey. Irregular structures are defined as the structures which does not have effective earthquake behavior due to its insufficient behavior under earthquake. In general, irregularities are defined in two main groups as irregularities in plan and in elevation. Irregularities in elevation are classified as soft story, also known as, rigidity irregularities and weak story, defined as that, the shear strength at any story is less than 80% of the story above of it in accordance to Turkish Design Code (TDC 2007), non-continuity for structural elements. In general, it can be observed many of the structural deficiencies and mistakes such as non-ductile details, soft and weak stories, short columns, strong beams–weak columns, large and heavy overhangs, poor concrete quality, wrong splicing of bars, and insufficient lateral reinforcement (Inel 2008, Irtem 2007, Ay *et al.* 2006). Moreover, incorrect site applications, due to the lack of supervision and inconsiderate contractors, are among the constructional deficiencies facing Turkey (Tankut 1999).

Masonry walls are used commonly as infill walls in both the interior and exterior RC frames. The material of the masonry infill is the main variant, ranging from natural stones to bricks and blocks. It is demanding to design these building structures for satisfactory structural behavior (Demir and Sivri 2002). In general, in seismic analyses masonry infill walls are considered by its weights not by interaction between walls and frames. The infill walls are commonly seen in Turkey and it is very important to determine the effects of infill walls to structural behavior (Korkmaz *et al.* 2007).

In the present study, the effects of masonry walls on structural damage probability are investigated with probabilistic assessment. For probabilistic assessment, fragility curves, sketched analytically are used. 5 and 8-story R/C buildings with and without masonry walls are dimensioned and analyzed. First, pushover curves are sketched for the representative buildings. Aftermath, time history analyses are carried out with different 240 real ground motion data and displacement demands are determined. Maximum relative story displacements are selected as damage parameter. Damage levels are defined with determined displacement demands. Peak ground velocity (*PGV*) is selected as ground motion parameter. The generated fragility curves are expressed in the form of two parameter lognormal distribution functions and the mean and the standard deviation parameters of this distribution are defined using the smallest square root methodology.

Fig. 1 Typical RC constructions in Turkey

2. Residential buildings in Turkey

The metropolitan cities in Turkey such as Istanbul and Izmir are under high seismic risk. Population in these cities constantly increases over the last three decades due to migration from less developed regions of the country. During this population boom, most of the buildings that are typically low to mid rise reinforced concrete structures with infill walls, were generally not designed according to the current seismic design code, which was 1975 version of the Turkish Design Code (TDC 1975). Also, the supervision in the construction phase of these buildings was not adequate (Erberik and Cullu 2006). Therefore, the majority of this building stock is generally composed of low-engineered buildings (Erberik 2008). An example of typical RC residential building is shown in Fig. 1, which is ubiquitous in Turkey.
Many such buildings were subjected to strong ground motions during recent earthquakes in Turkey (Ozcebe *et al*. 2004). The large number of casualties due to heavy damage or collapse of this kind of buildings demonstrates the vulnerability of such reinforced concrete buildings during strong earthquakes. It is probable that such devastating earthquakes will occur in the future. To avoid the future earthquakes with less damage and losses, determination and assessment of seismic vulnerability of these buildings considers quite imperative.

3. Recent earthquakes in Turkey and infill wall effects

In August and November 1999, two major earthquakes hit the northwestern region of Turkey resulted with more than 50,000-property and 20,000-live lost. Typical building damages following the 1999 Marmara earthquakes are shown in Fig. 2. After this countrywide disaster experience, researchers focused on existing buildings and carried out various projects. Also, over the last two decades disaster mitigation programs have gone through at paradigm shift. As a result of these conducted research works, most of the existing buildings were tagged as suspicious in terms of structural behavior satisfaction level for future earthquakes. After 1999 Marmara earthquakes, Turkey have experienced other major earthquakes in 2003 in Bingol and two recent devastating earthquakes in east part of the country: March 2010 Elazig and October 2011 Van earthquakes. The results were similar with the 1999 Marmara earthquakes. Structural damages due to the infill walls are typical in the experienced earthquakes.

In Elazig earthquake (M_w = 6.1) on March 8, 2010, 42 people lost their lives and 137 people were injured. The earthquake caused major structural damage in residential and school buildings. A significant percentage of the building stock in the region is low-story masonry buildings constructed with adobe and stone walls, with low quality mortar. Occurred damages in infill walls are similar to those observed in previous earthquakes.

In October 23, 2011 Van earthquake (M_w = 7.2), the disaster is at high level. The buildings behaved insufficient for the experienced earthquake, even the earthquake was not a design earthquake defined in Turkish Design Code. After the main earthquake, in November 9, 2011 Van Edremit earthquake (M_w = 5.6) was occurred as a following earthquake. Due to the earthquake, more than 2000 buildings collapsed and more than 4000 buildings got severe damage. More than 600 people lost their lives and more than 4000 people were injured.

Fig. 2 Typical damages in R/C buildings after 1999 Marmara earthquakes in Turkey

Fig. 3 Typical damages after 2011 Van earthquakes in Turkey

Fig. 4 Infill wall behavior after Van earthquakes in Turkey

In Van Region, buildings in city center are R/C buildings and the ones in villages are low-story masonry buildings constructed with adobe and stone walls. In typical R/C buildings beside the slab, columns and connecting beams, exterior and interior unreinforced brick masonry infill walls are placed in the residential buildings. Soft stories are also observed in the region. Building damages in Van earthquakes are shown in Fig. 3. In Fig. 4, infill wall behavior is shown.

4. Turkish design codes

2007 version of Turkish Design Code is currently valid in design (TDC 2007). Up to now, 8 different versions of Turkish design codes have been developed. The first development in the field of design codes in Turkey was recognized after Erzincan Earthquake in 1939. The design codes have been developed due to insufficiencies in design and construction after experienced countrywide disasters. Temporary regulations started to be applied in 1940s and the first code took effect in 1944. In time, with experiencing severe earthquakes that led to thousands of live lost and collapse of buildings, seismic codes were revised frequently. 1949, 1953, 1962, 1968, 1975, 1998 are some of the versions. Current design code was developed in 1998 (TDC 1998), and revised and a section for existing buildings was added in 2007 to evaluate the performance of existing buildings (TDC 2007).

Buildings, built according to 2007 version may provide a sufficient resistance to earthquake; however, older buildings, especially those built before the 1998 version, suffer severe damage and destruction from earthquakes due to insufficient structural resistance which was experienced in previous earthquakes. Based on the results of field investigations and research work, the main reason for structural damage observed in R/C buildings is simple construction errors and missing link in structural behavior and labor.

The earthquakes are uncertain events. However, it is a fact that the earthquakes are frequently occurred in Turkey. The codes developed on the basis of science and past experiences mainly aim to provide the life safety level. Although the codes were revised several times, occurred earthquakes in Turkey have caused severe damages in the buildings while the other parameters played an important role such as labor, material quality, soil conditions, and etc. which affect the structural performance of the buildings. Therefore, it might not be accurate to make a definite statement about the effects of Turkish design codes on structural behavior yet.

5. R/C building representative models and nonlinear analysis

In analytically derived fragility curves damage distributions obtained from analyses of buildings under earthquake loads are used. Due to this requirement, nonlinear behavior of structural component must be taken into consideration in the analyses. In the analyses of this study, 5 and 8 story representative buildings are used. Story height for the buildings is considered as 3 m for both structures. Infill walls are located in 1-2, 3-4 and 5-6 vertical axes, and in A-B, C-D and D-E horizontal axes of the plan views of representative buildings. One span in both directions is considered without infill walls. Such an arrangement has been done to represent the window and door openings in the building. An asymmetrical arrangement adopted in the plan view for infill walls does not cause any significant torsion in the building. There is only one span without infill wall in both directions of the buildings and also thicknesses and rigidities of the walls in both directions are the same and the representative buildings are perfectly symmetrical. Walls are continues from bottom to top of the buildings. For 5-story building, beams dimensions are 25 × 50 cm and columns dimensions are 50 × 50 cm. For 8-story building, beams are 30 × 60 cm, columns are 60 × 60 cm and shear walls are 30 × 210 cm. In Fig. 5, plan view of 5 and 8-story R/C buildings are given. Elevation views of 5 and 8-story buildings are shown in Fig. 6.

The characteristic compressive strength of concrete is assumed to be 20 MPa and the characteristic yield strength of longitudinal bars is assumed to be 420 MPa. Since most of buildings in Turkey have poor confinement, the effect of confinement is not taken into consideration in modeling phase. Although, in the design of the representative buildings, all design requirements of the Turkish Design Code have been satisfied, in the analyses, only longitudinal reinforcements are considered in computing structural member capacities to consider the worse case scenarios. When confinement effects are considered, the ductility and moment capacities increase.

Plastic hinge theory is used to define nonlinear behavior of structural materials. In this theory, plastic deformations are lumped at plastic hinges. At other sections between plastic hinges, material behavior is accepted as linear elastic. It is assumed that, plastic behavior is assigned with one-dimension bending moment for beams, two-dimension bending moment and axial force interaction for columns. Therefore, M3 hinges are used for beams and PMM hinges are used for columns and they are assigned at both ends of beams and columns.

Shear walls are modeled as statically equivalent columns with same sections. To reveal the real behavior of shear walls under bending effect, rigid beams are used. Moment-plastic rotation relation of beams, columns and shear walls are assumed as strengthening rigid plastic. Plastic hinge information of members as plastic rotations, -plastic moments (force-deformation behavior of plastic hinges) are taken from ATC 40 (ATC 1996). Cracked section rigidity of beams, columns and shear walls are taken as recommended in FEMA 356 (FEMA 2000). Shear capacity of structural members are checked to avoid brittle failure in the analyses.

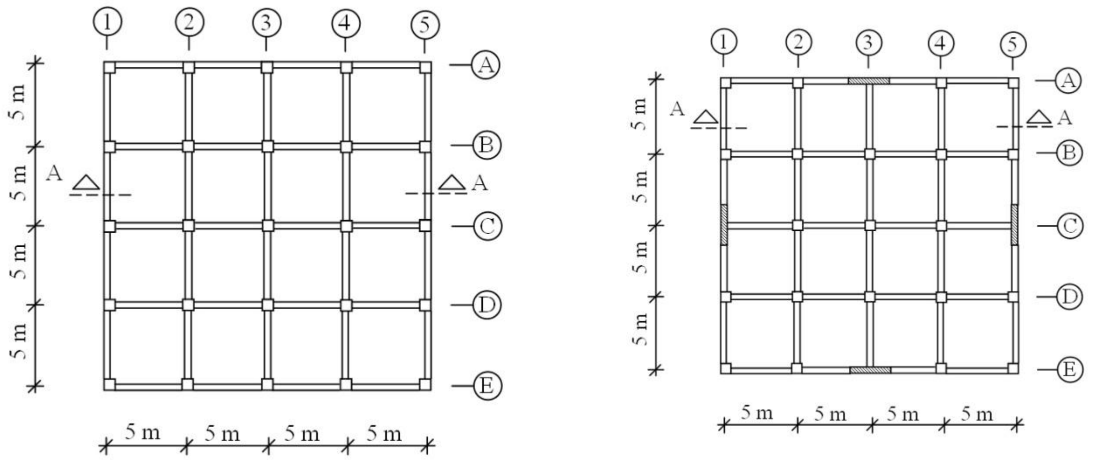
Infill walls are modeled with equivalent diagonal compression struts as shown in Fig. 7. The equivalent strut assumed to have the same thickness and modulus of elasticity as the infill wall it represents. The thickness of the masonry infill walls is considered as 20 cm, since it is the most commonly used masonry wall thickness in Turkey. Diagonal strut bandwidth is defined according to FEMA 356 by using Eqs. (1) and (2).

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{\inf} \tag{1}$$

$$\lambda_1 = \left[\frac{E_{me}\, t_{inf}\, \sin 2\theta}{4 E_{fe}\, I_{col}\, h_{inf}} \right]^{0.25} \tag{2}$$

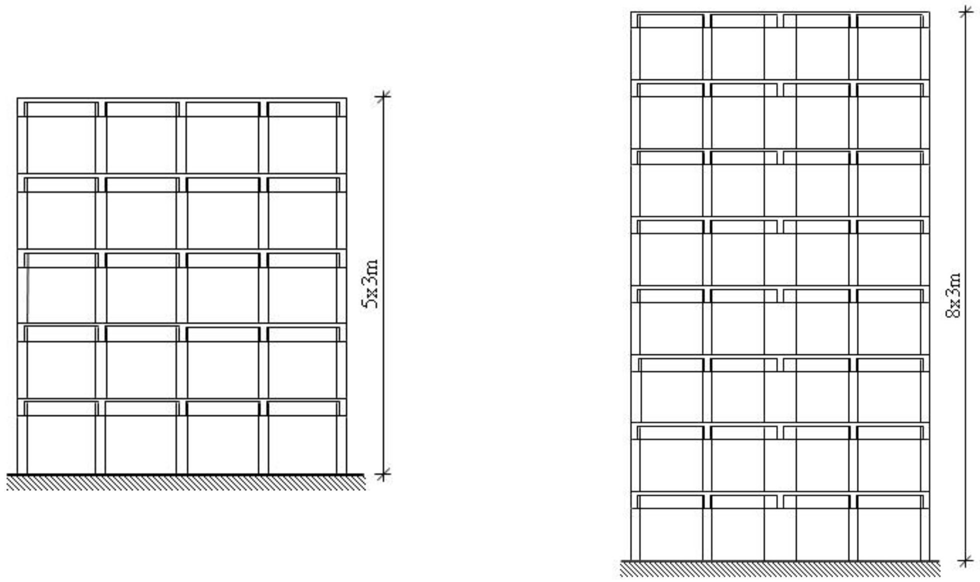
where h_{col} is column height between centerlines of beams and h_{inf} is height of infill wall. E_{fe} and E_{me} are expected modulus of elasticity of frame and infill wall materials', respectively. I_{col} is moment of inertia of column, L_{inf} is infill wall length, r_{inf} is diagonal length of infill wall, t_{inf} is thickness of infill wall and equivalent strut, λ_1 is coefficient used to determine equivalent band width of infill strut and θ is angle whose tangent is the infill height to length aspect ratio (in radians).

N-Δ_p relationship of equivalent strut is given in Fig. 7 (Hanoglu 2004, Irtem *et al*. 2005, Irtem *et al*, 2007). In this figure, N_{max} and N_{min} are maximum and minimum compression strengths of



(a) 5-story representative building (b) 8-story representative building

Fig. 5 Plan view of 5 and 8-story representative buildings



(a) 5-story representative building (b) 8-story representative building

Fig. 6 Elevation view of 5 and 8-story representative buildings

infill wall, respectively and N_c is cracking strength of infill wall. Plastic deformations related with these strengths are given in horizontal axis of the same figure. In the analysis, modulus of elasticity of infill walls is assumed to be 6000 MPa.

Building models' nonlinear analyses as pushover analyses are realized with SAP2000 nonlinear analysis program (Wilson and Habibullah 1998). Since the building is symmetrical, pushover curves at X and Y directions are same. In pushover analysis, the behavior of building is characterized by a pushover curve that represents the relationship between the base shear force and the lateral displacement of the roof. This is a general trend in practice (Korkmaz and Karahan, 2011). Pushover curves of the buildings are given in Fig. 8 and Fig. 9. In analyses, 5 story building is abbreviated as 5S and 5S-I, respectively due to absence or presence of infill walls. Similarly, 8 story building with shear walls is abbreviated as 8SS and 8SS-I. In Table 1, effective periods (T_e) and base shear coefficients (V_T/W) are given. As seen in Table 1, when the infill walls are considered in the analyses, periods of the representative buildings are shorten. Masses of floors are calculated from gravity loads (the dead loads plus 30% of the live loads) and are assumed as lumped at each story level.

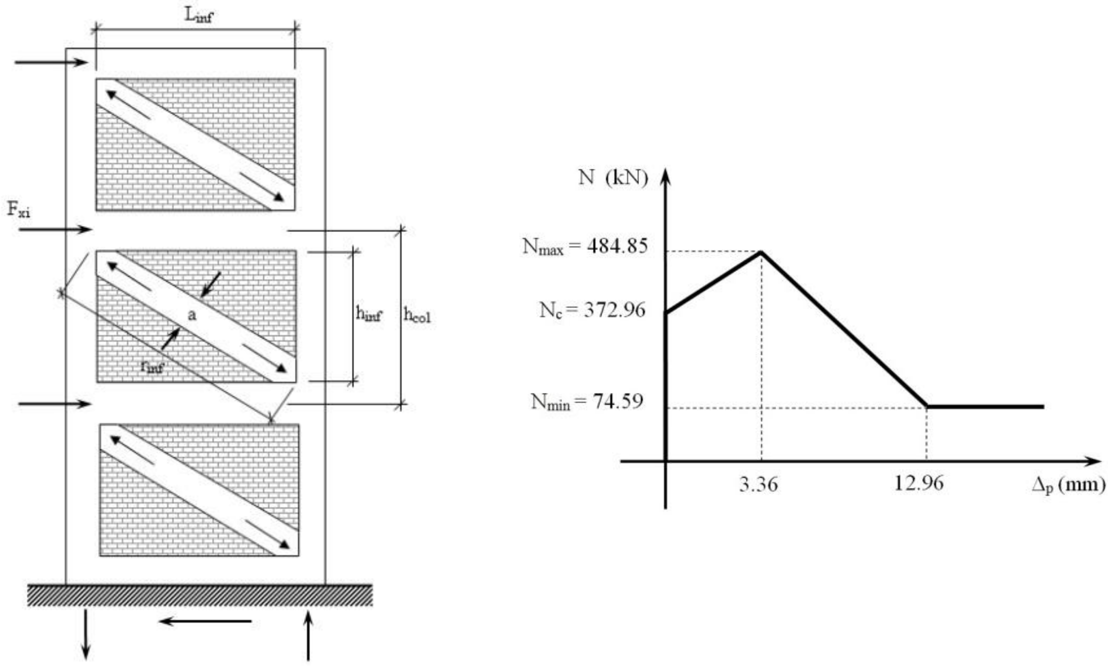


Fig. 7 Modeling and N-Δp relationship of Infill Walls

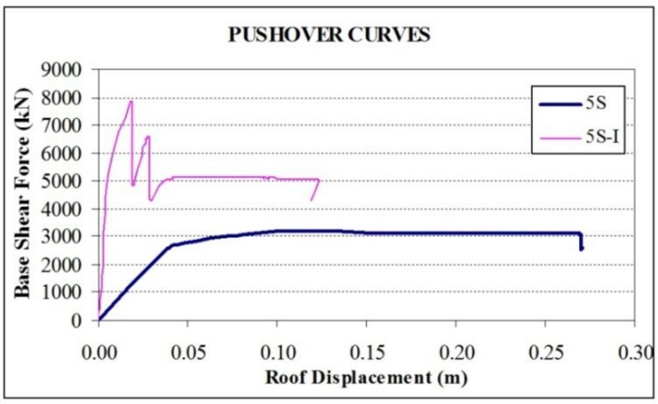


Fig. 8 Pushover curve of 5-story building

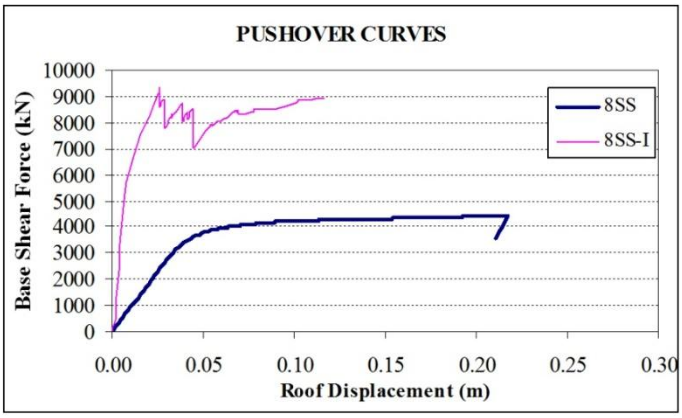


Fig. 9 Pushover curve of 8-story building

Table 1 Effective periods and base shear coefficients

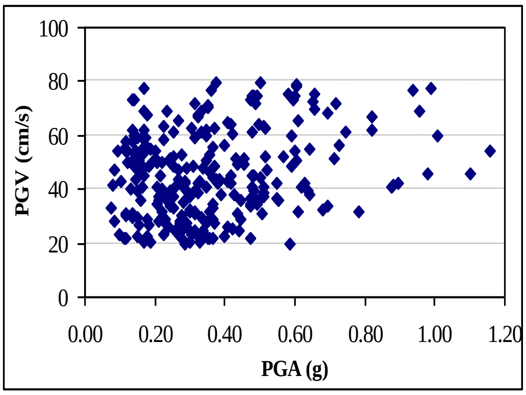
Model	T_e (s)	V_T/W
5 Story (5S)	0.80	0.12
5 Story with Infill walls (5S-I)	0.24	0.19
8 Story (8SS)	0.87	0.09
8 Story with Infill walls (8SS-I)	0.35	0.16

6. Fragility analyses

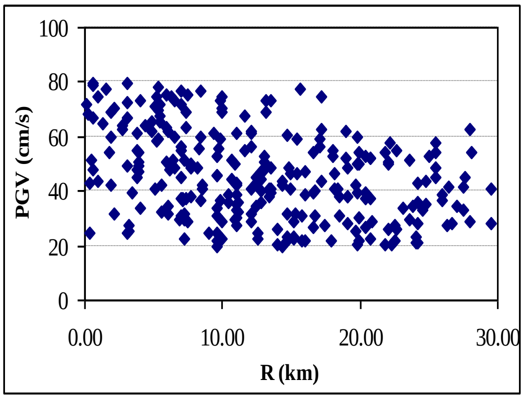
Fragility curves are sketched in four ways as ampirical, analytical, judgmental and hybrid according to whether the damage data used in their generation derives mainly from observed post-earthquake surveys, analytical simulations, expert opinion or combinations of these, respectively (Rossetto and Elnashai 2003). Amprical way is based on experimental earthquake data (Yamazaki *et al*. 2000, Basoz and Kiremidjian 1997, Shinozuka *et al*. 2000a, Rossetto and Elnashai 2003, Kircher *et al*. 1997). Analytical way is based on structural analysis results (Karim and Yamazaki, 2001, Shinozuka *et al*. 2000b, Erberik and Elnashai 2004, Kirçil and Polat 2006, Akkar *et al*. 2005). Structural analysis related with earthquake ground motion data and spectral parameters (Erberik and Elnashai 2004, Kircil and Polat 2006).

Fragility curves (or functions) are the essential tools for seismic loss estimation in built environments (Akkar *et al*. 2005). They describe the conditional probabilities of sustaining different degrees of damage at given levels of ground motion (Singhal and Kiremidjian 1996). There are several ways to generate fragility curves analytically. Although, any earthquake or spectral intensity can be chosen as earthquake parameter, which constitute abscissa of fragility curve, especially for mid-size R/C buildings, *PGV* could be used as a parameter for fragility analysis (Karim and Yamazaki 2001, Ay *et al*. 2006, Akkar *et al*. 2005).

In the fragility analysis part, regarding with time history analysis results as displacement demands, fragility curves are sketched for representative buildings. For dynamic time history analysis, 240 ground motion data belongs to 20 different earthquakes (PEER 2012). These



PGA-PGV



PGV-R

Fig. 10 Interaction of *PGA* to *PGV* and of *PGV* to *R*

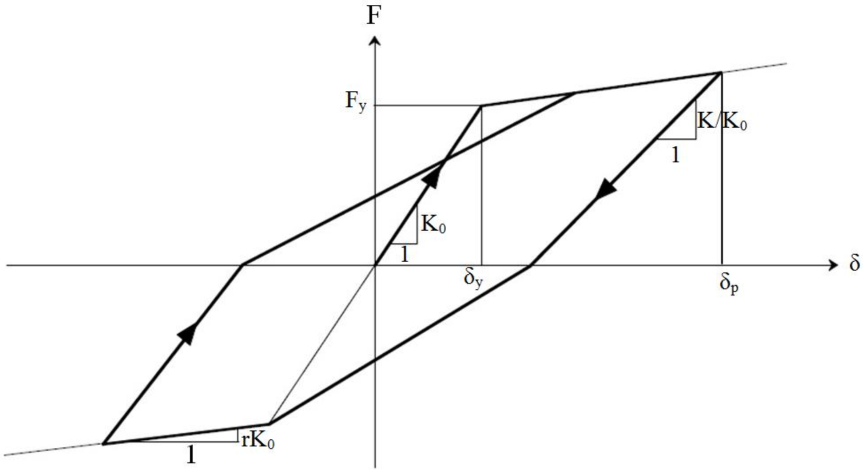
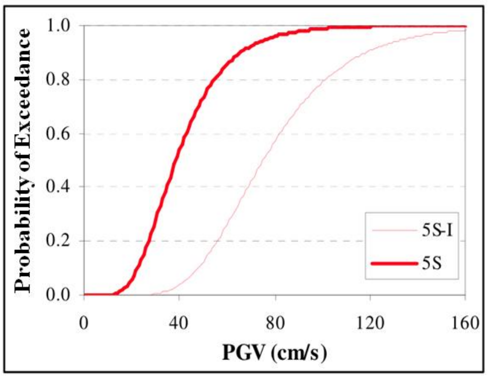
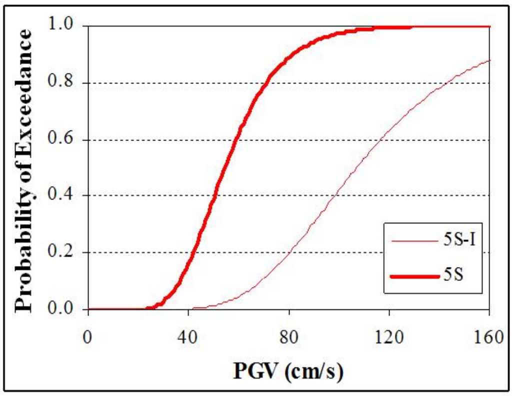


Fig. 11 Force-displacement relationship of Takeda hysteretic model

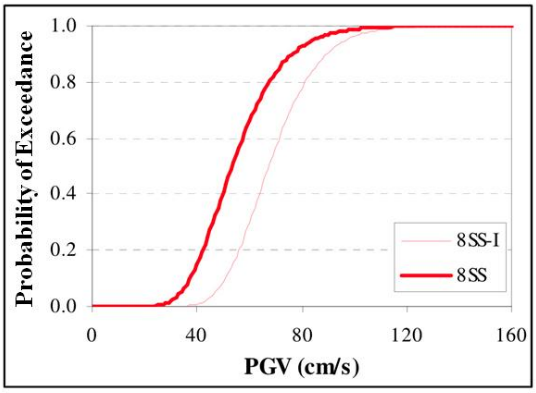


(a) Moderate

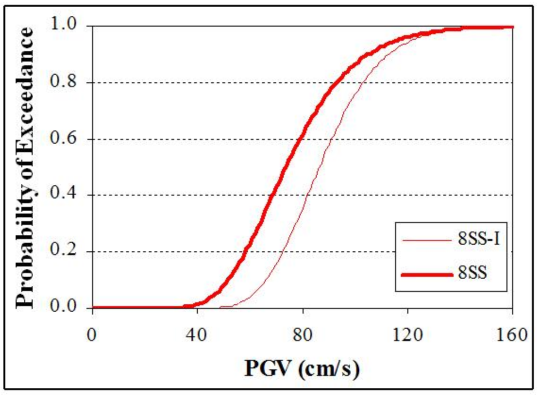


(b) Extensive

Fig. 12 Fragility curve for 5-story building



(a) Moderate



(b) Extensive

Fig. 13 Fragility curve for 8-story building

Table 2 Damage levels and limit stages (Erberik and Elnashai 2004)

Limit state	*DR* (%)
Slight	0.1
Moderate	1.0
Extensive	2.0
Collapse	3.5

earthquakes' magnitudes are at between 5.4 and 7.1. The earthquakes are selected with R < 30 km. In Fig. 10, interactions of Peak Ground Acceleration (*PGA*) to Peak ground Velocity (*PGV*) and of Peak Ground Velocity (PGV) to distance (R) are given.

In time history analyses, Takeda hysteretic model is considered for nonlinear force-displacement relationship of R/C members. In this model, degradation of the stiffness due to increasing damage can be considered (Takeda *et al*. 1970). Force-displacement relationships of Takeda hysteretic model is shown in Fig. 11. In this figure, K_0 is initial stiffness, rK_0 is post yield stiffness, δ_y is yield displacement, δ_p is peak displacement and F_y is yield force.

In performance based seismic design, quantifying seismic damage of complete structures by damage indices is one of the fundamental issues (Lu *et al*. 2011). Moreover, quantification of seismic damage plays an important role to sketch fragility curves. In the present study, damage index is assigned as interstory drift ratio parameter (*DR*) (Erberik and Elnashai 2004). Four different damage levels are given in Table 2, although only two of them are used in this study.

The sequence of plastic hinge formation for each load increment is achieved from pushover analyses of representative buildings. Interstory drift ratios related with damage levels are gained from lateral story displacements. The infill walls fail at a small interstory drift ratio, at around 0.1%. When the intersory drift ratio is at around 1%, plastic hinges are formed at both ends of many beams of the buildings. For 1% interstory drift ratio corresponding to moderate damage, plastic hinges are also formed at bottom end of some columns at the base level. However, plastic rotation values of the hinges are at insignificant levels. When the interstory drift ratio is approximately 2%, plastic hinges are formed at all of the bottom ends of the base columns and plastic hinges at the both ends of beams reached to the rotation capacity. Hence, the building became near to collapse. For 2% of interstory drift ratio, the damage level of the building is assumed to be extensive.

Damage values at each damage levels are determined with different *PGV* values. Fragility levels are assumed as in lognormal distribution. For determination of parameters of mean and deviation of lognormal distribution, the Smallest Square Root Method is applied. With Eq. (3), fragility level is defined for each damage level and in different *PGV* values.

$$F_{ij} = \Phi\left(\frac{\ln(DR_i) - \mu_{\ln(DR)}}{\sigma_{\ln(DR)}} \middle| PGV = v_j \right) \tag{3}$$

In Eq. (3), Φ is standard normal distribution function, μ and σ are mean and lognormal standard deviation values of damage parameters, respectively. In this study, fragility curves for representative buildings are sketched depending on moderate and extensive damage levels. In Figs. 12 and 13, fragility curves are given.

7. Conclusions

The present study reveals the poor seismic performance of RC buildings with and without masonry infill walls, and analysis is underlying design parameters causing such performance. Therefore, probabilistic structural seismic assessment is presented with fragility analysis.

In nonlinear analysis part, first, pushover analyses are carried out to achieve pushover curves and define damage states. In the pushover analyses, failure mechanism of the representative buildings is also achieved by observing the sequence of plastic hinge formation for each load increment. The infill walls fail at a small interstory drift ratio, at around 0.1%. When the intersory drift ratio is at around 1%, plastic hinges are formed at both ends of many beams of the buildings. For 1% interstory drift ratio corresponding to moderate damage, plastic hinges are also formed at bottom end of some columns at the base level. However, plastic rotation values of the hinges are at insignificant levels. When the interstory drift ratio is approximately 2%, plastic hinges are formed at all of the bottom ends of the base columns and plastic hinges at the both ends of beams reached to the rotation capacity. Hence, the building became near to collapse.

Later, time history analyses were carried out with 240 ground motion data belongs to 20 different earthquakes. These earthquakes' magnitudes are at between 5.4 and 7.1. After nonlinear analyses, fragility curves are sketched for two different damage levels. Fragility curves are shown in Fig. 12 and Fig. 13. Considering these fragility curves, the following conclusions can be stated:

1) The presence of infill walls increases the lateral stiffness of the buildings. It is an evident that the slope of pushover curves increases as the infill walls are considered in the analyses (As seen in Fig. 8 and Fig. 9). After failure of infill walls, the slope of pushover curves decreases and a drop in the strength of the building occurs.

2) It is obvious in Fig. 12, probability of structural damage is decreasing for the 5-story building with infill walls. For example, when *PGV* is 60 cm/s, moderate damage probability is 90% and 30% for 5-story building with and without infill walls respectively. Similarly, when *PGV* is 80 cm/s, extensive damage probability is 90% and 20% for 5-story building with and without infill walls, respectively.

3) Probability of damage is also decreasing for the shear wall buildings with infill walls (Fig. 13). For example, when *PGV* is 60 cm/s, moderate damage probability is 80% and 40% for 8-story

building with and without infill walls respectively. Similarly, when *PGV* is 80 cm/s, extensive damage probability is 70% and 35% for 5-story building with and without infill walls, respectively.

4) The last two results indicate that the presence of infill walls decreases the damage probabilities and improves the seismic behavior of R/C buildings. This fact should be taken in the consideration in the design of new buildings and the presence of infill walls should considered in the analyses by representing them with realistic analytical models.

5) Fragility curves of 8-story buildings with and without infill walls are closer to each other when compared with generated fragility curves of 5-story buildings. This result indicates that, the effect of infill wall, which decreases the structural damage probability, is more specific in R/C frame buildings according to buildings with shear walls.

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