Computers and Concrete, Vol. 6, No. 5 (2009) 403-419

DOI: http://dx.doi.org/10.12989/cac.2009.6.5.403

Failure analysis of reinforced concrete frames with short column effect

Naci Caglar† and Mahir Mutlu‡

Department of Civil Engineering, University of Sakarya, Sakarya 54187, Turkey

(Received December 4, 2008, Accepted August 24, 2009)

Abstract. Short column effect is cause to failure of columns which may result in severe damages or even collapse during earthquakes. The scope of the study is mainly to reveal the effect of short column on the holistic behaviour of the buildings. The nonlinear analysis of 31 different frame buildings containing short column problem are carried out using finite element method. The finite element models were selected by 2 bays and 3 stories. Since the short columns are generally seen in the first storey of the buildings, in the study, they are only constructed in the same storey. The adverse effect of the short column on the response of buildings was shown in terms of the total load factor and displacement capacity of building. The response of buildings in terms of ground storey displacements is presented in figures and discussed. It is revealed that if the window openings are constructed along the bays, the total load capacity is decreased 85% compared with reference model in which all of bays are filled with infill walls.

Keywords : short column effect; RC frame; failure of columns; nonlinear analysis; finite element analysis.

1. Introduction

Turkey is located on one of the most active earthquake zone which has earthquake periods quite often with shortest return periods. During the last century, more than twelve major earthquakes with minimum magnitudes 7 (Mw) caused significant casualties, severe damages to a lot of structures and lifelines in Turkey. In the last century, approximately 500,000 building collapsed and were heavily damaged (Arslan and Korkmaz 2007). In Turkey, the construction quality of RC structures varied widely.

During the earthquake, the short column effect is one of the main reasons to cause the failure of columns which leads the building to collapse. Short columns may be generally developed due to window openings which is put in infill walls between columns. Because of the short column bahaviour, column becomes stiffer and more rigid in bending. Thus, columns will take more bending moment and shear forces that will cause failure in shear (Dagangun 2004).

Cagatay (2005) investigated the failure of an industrial building due to short column effect in the industrial zone of Adana, Turkey. During the Adana-Ceyhan earthquake on June 27, 1998, all the columns on the outer side of the building failed because of short column effect. He reported the second storey of the building moved about 10 cm with respect to the base. He performed a dynamic analysis of the building. He presented suggestions to eliminate the short column effect.

† Assistant Professor, Corresponding author, E-mail: caglar@sakarya.edu.tr
‡ Ph. D. student

In a number of works (Guevara and García 2005, Nikos *et al.* 2008, Negro and Verzeletti 1996, Tegos and Penelis 1988) it has been studied the effect of short column on the seismic performance of RC frames. Short columns are a construction practice that is often encountered in RC buildings, mainly industrial. Short columns at the ground storey of the structures are prone to brittle shear failure which may result in severe damages or even collapse because of the poor ductility during earthquakes (Guevara and García 2005).

In this study, the nonlinear analyses of 31 of different models, two dimensional RC moment resisting frame buildings, were carried out. The aim of the study is to investigate the short column effect on the failure of RC moment resisting frame buildings during the earthquake. The finite element software program, LUSAS (2006), is used to simulate the response of reinforced concrete (RC) frame building with short column problem. The short column affects are generally seen in the first storey of the buildings due to architectural usage or structural arrangements. Thus in this study, short columns are only constructed in the first storey in finite element models.

2. Short column problems

If the infill walls in a frame in a reinforced concrete structure are built shorter than length of neighbouring columns, those columns are said to be short columns. Because of the absence of enough gaps between the columns and the infill wall, the columns can not deform laterally under the lateral loads. In same storey there exist short columns and regular which are not connected to infill walls. That is why, when the floor slab moves horizontally under the lateral loads from an earthquake, the upper ends of both of short columns and regular columns undergo the same displacement. While the regular columns deform over the full height, the short columns deform by the full amount over the short height adjacent to the window openings. In this case, excessive shear forces occur in the short column and this causes the failure of columns which leads the building to collapse (Fig. 1).

The shear force, V, can be calculated by the equilibrium of the short column as;

$$V = \frac{M_t + M_b}{L_s} \tag{1}$$

where M_t and M_b are the moments at the top and bottom of the short column, respectively, and L_s is the length of the short column (Fig. 2)

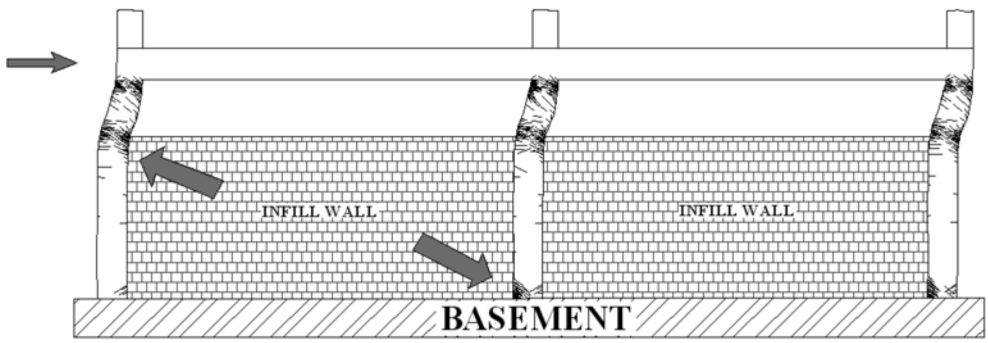


Fig. 1 The short column behaviour in frame structures

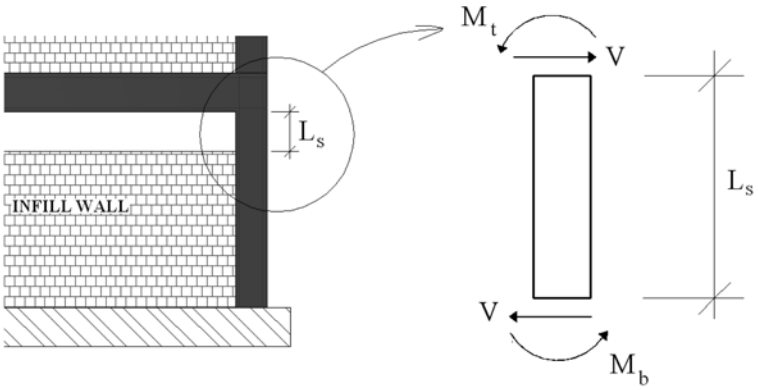


Fig. 2 Moments and shear force acting on a short column

Fig. 3 Shear failure at short column (Erdik *et al.* 2003)

In the design of the real buildings, the mathematical model contains only beams, columns and slab. Thus, the effect of infill walls on the design of reinforced concrete structures has been ignored. The real behaviour of the structure with infill walls during an earthquake differs from this case. If the short column phenomenon in the reinforced concrete structures is not considered during the design, the shear forces in the columns may cause collapse of the building (Fig. 3).

3. Numerical application

In this study, the nonlinear analyses of two dimensional RC moment resisting frame buildings were carried out to investigate the effects of short column on its response by using finite element software program, LUSAS (2006). The finite element models were constructed to simulate the

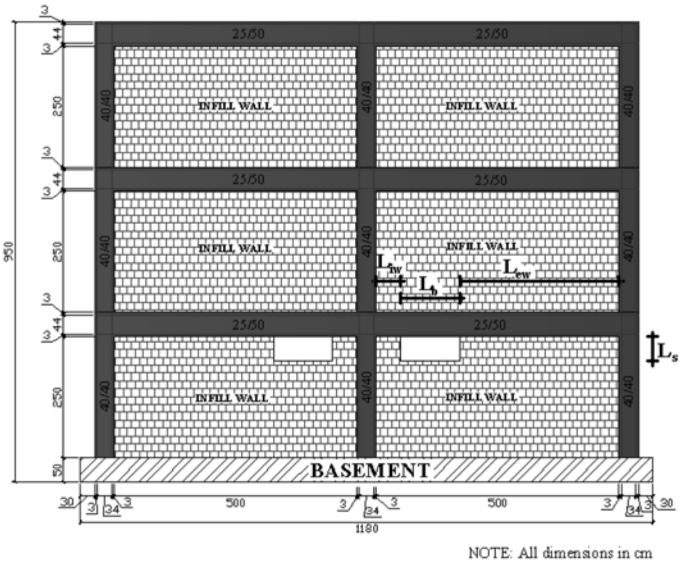


Fig. 4 The general form of two dimensional RC moment resisting frame building (all dimensions in cm)

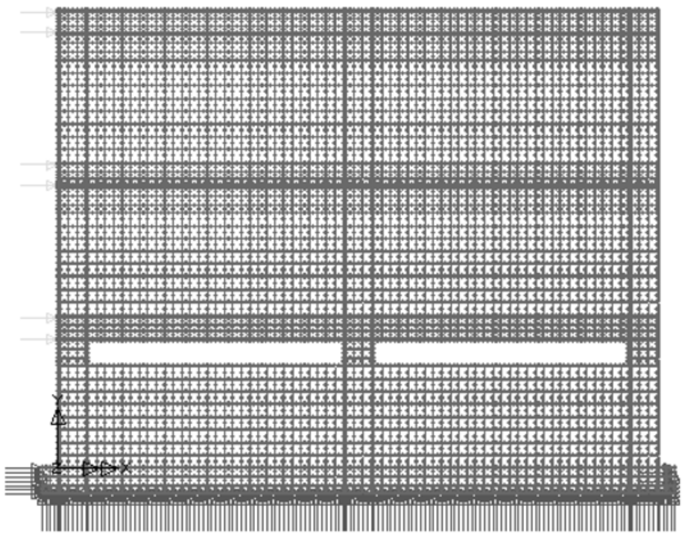


Fig. 5 The general FEA mesh of 2D frame models

response of RC frame building which had short column problem. The models were selected by 2 bays and 3 stories. The selected parameters consisted of the height of the window opening (L_s), the length of the window opening (L_b), the distance (the length of the infill) between the interior column and window opening (L_{iw}), the distance (the length of the infill) between the exterior column and window opening (L_{ew}) (Fig. 4).

Finite element model is constructed by using regular and fine mesh. Boundary condition for models is fully fixed through the basement (Fig. 5).

In all of the reinforced concrete structures, dead load and live load are taken as p=2.0 kN/m^2

and q=1.50 kN/m^2, respectively. The sections of structural elements are rectangular and their dimensions are kept constant for all stories. Columns are 40 cm × 40 cm, beams are 25 cm × 50 cm and infill walls are 13.50 cm in dimensions. The height of stories is 300 cm and the depth of the basement is 50 cm for all structures. The concrete cover is 3 cm for all reinforced concrete elements. The cross sectional areas of reinforcement on the bottom and on the top face of the beam sections are 452 and 226 mm^2, respectively. The cross section of columns is taken as 1230 mm^2. It is assumed that the concrete and reinforcement are perfectly bonded. In this study the effects of any shear reinforcement is ignored. A four-node quadrilateral plane stress shell element is used to model the concrete structure. Two dimensional quadratic, isoparametric bar elements are used to model the reinforcement. A nonlinear concrete cracking material model is applied to the plane stress shell elements and a von Mises metal plasticity is applied to the reinforcement bars. Modulus of elasticity for concrete is taken as $E_c = 4750\sqrt{f_c}$ (Ersoy and Ozcebe 2001). The mechanical properties of the materials are presented in Tables 1-2. For all of the infill walls values of the modulus of elasticity (Young's module), E=7800 N/mm^2; Poisson's ratio, $\nu = 0.15$; the mass density, ρ=8×10^{-6} N/mm^3 and the tensile strength, f_t=0.10 N/mm^2 are assumed. The behaviour of concrete under tensile and compressive stress is shown in Fig. 6, respectively. The uniaxial tensile strength of steel reinforcement bar is modeled according to Fig. 7. Also isotropic hardening is assumed for steel bars.

A unit of concentrated load has been applied and the load factor in the nonlinear control has been used to control the magnitude of loading. The concentrated loads have been applied to the points at

Table 1 The mechanical properties of concrete

Young's modulus, E (N/mm^2)	26000
Poisson's ratio, ν	0.20
Mass density, ρ (kg/m^3)	250
Compressive strength, f_c (N/mm^2)	30
Tensile strength, f_t (N/mm^2)	3.158
Strain at peak compressive stress, ε_{cp}	0.0022
Strain at end of compressive softening curve, ε_{co}	0.0035
Strain at end of tensile softening curve, ε_{to}	0.0030

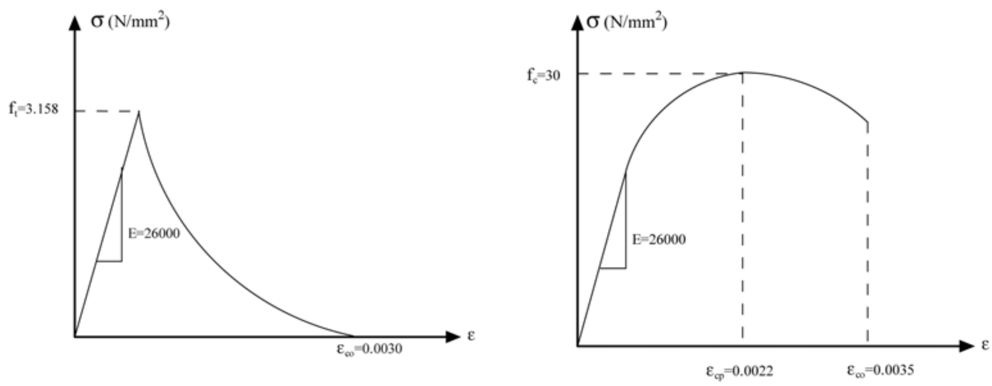


Fig. 6 Tensile and compressive behaviour of concrete (LUSAS 2006)

Table 2 The mechanical properties of the materials of the steel reinforcement bar

Young's modulus, E (N/mm^2)	210000
Poisson's ratio, ν	0.30
Initial unixial yield stress, f_y (N/mm^2)	300
Ultimate yield stress, $f_{y\ max}$ (N/mm^2)	340
Hardening gradient, E_{rp} (N/mm^2)	2154
Strain at end of hardening curve, (N/mm^2)	0.020

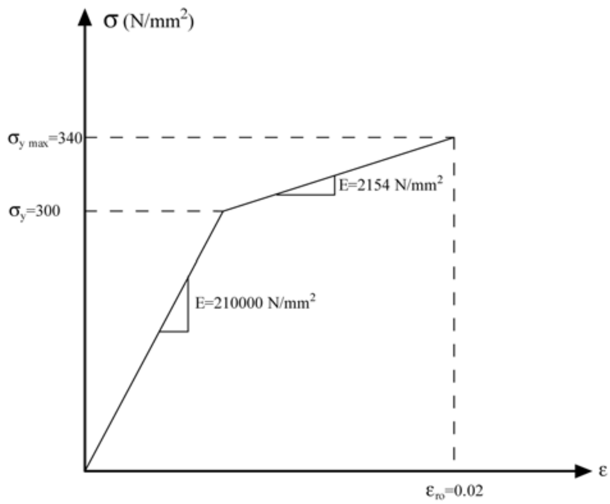


Fig. 7 Tensile and behaviour of steel reinforcement bar (LUSAS 2006)

Table 3 The general form of loading conditions

	Starting Load Factor	Max. Change in Load factor	Max. Load factor
Load Factor(kN)	50	100	600

the top and bottom of the beam at story levels. The loading scheme for the frames is given in Fig. 8. The nonlinear analysis has been carried out from a starting loading factor (*LF*) of 50 kN to a max loading factor of 600 kN in maximum change in load factor of 100 kN (Table 3).

4. Verification of the FEM model

The results from the numerical study are compared with the real short column damage to verify the validity of the finite element models (Fig. 9). A careful study of the results from Fig. 9 leads to an observation of fairly good agreement between the real problem and FEA model outcomes.

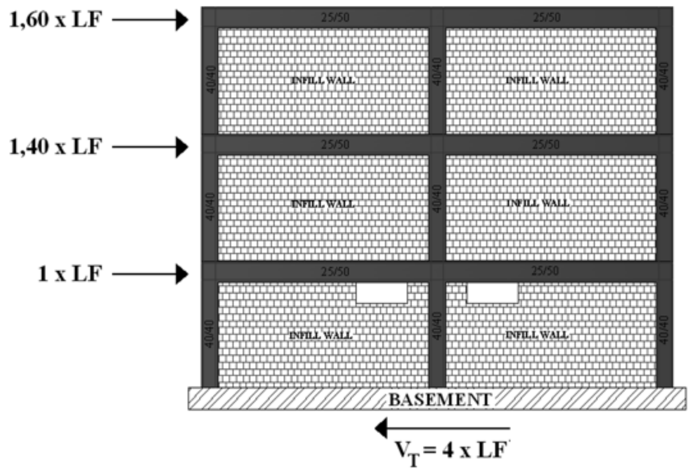
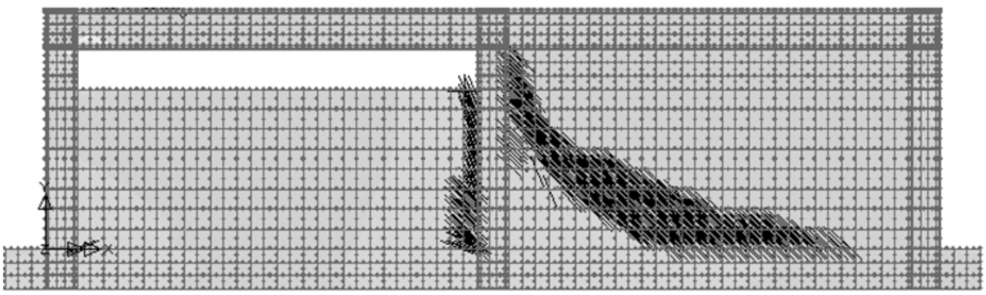


Fig. 8 The loading type of frame structures

a) short column damage



b) Finite Element Model

Fig. 9 The real short column damage and the simulation of finite element model

5. Results and discussion

In this study, 31 FEM models are constructed for nonlinear analysis. The response of frame building in terms of ground storey displacements is presented in figures. There are six main models (Model A, B, C, D, E, F) and five sub-models for each one. The load-displacements curves are plotted for five sub-models within a graph. Model H (Fig. 10) in which short column effect is neglected is selected as a reference model.

Table 4 Reference model H

Models	Windows in	L_{iw}, (cm)	L_b, (cm)	L_{ew} (cm)	L_s (cm)
Model H	none	500	0	0	0

L_s is the height of the window opening; L_b is the length of the window opening; L_{iw} is the distance between the interior column and window opening; L_{ew} is the distance between the exterior column and window opening

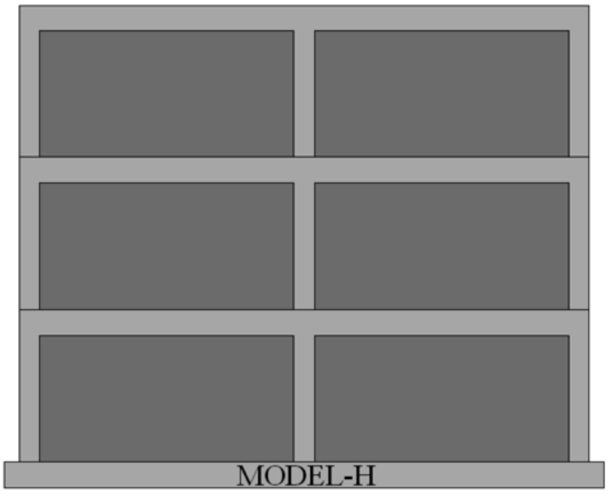


Fig. 10 Reference model H

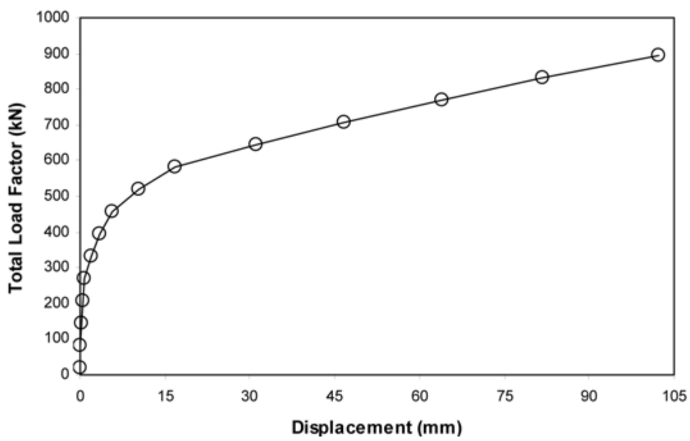


Fig. 11 The ground storey displacements of Model H, reference model

Fig. 11 shows ground story displacement, versus the total load factor. It demonstrates good ductile behavior, as expected.

5.1. Model A buildings

Window opening is located only in one bay while the other bay is filled with infill walls for Model A buildings (Fig. 12). The distance between window opening and exterior columns (L_{ew}) varied from 1 m to 3 m while the wall distance between window opening and interior columns (L_{iw}) is kept at 1 m constant (Table 5).

While the capacity of total load factor (TLF) is 900 kN in the reference model, this value at the Model A buildings decreases to about 450 kN. As shown in Fig. 13, the capacity of TLF at frame buildings containing window openings is generally 50% lower than reference building. The response of buildings in terms of capacity of TLF shows almost similar behaviour since infill wall is present between the columns and window openings for Model A buildings.

5.2. Model B buildings

The parameters, which are used to describe the form of short columns, are the same for Model B and Model A buildings. The only difference between these models is a presence of window

Table 5 The forms of short columns of model A frame buildings

Models	windows in	L_{iw}, (cm)	L_b, (cm)	L_{ew} (cm)	L_s (cm)
Model A1			100	300	
Model A2			150	250	
Model A3	one bay	100	200	200	50
Model A4			250	150	
Model A5			300	100	

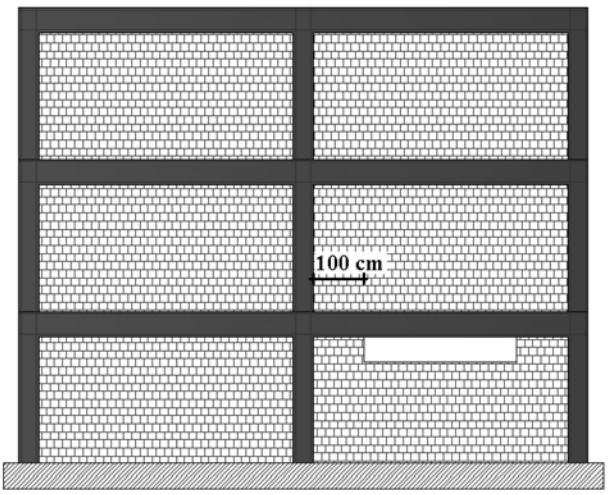


Fig. 12 The general form of window opening in model A buildings

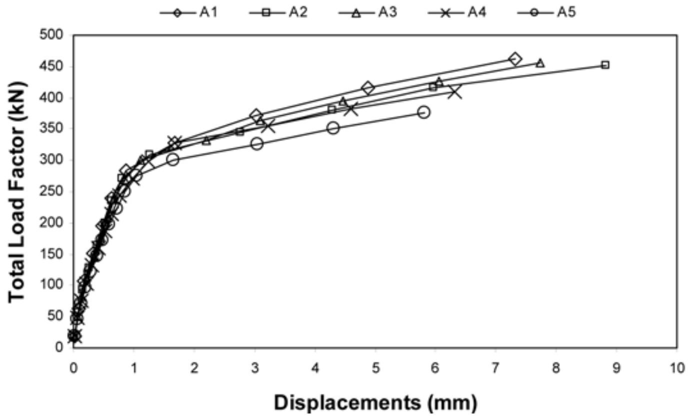


Fig. 13 The ground storey displacements of model A buildings

Table 6 The forms of short columns of model B frame buildings

Models	windows in	L_{iw}, (cm)	L_b, (cm)	L_{ew} (cm)	L_s (cm)
Model B1			100	300	
Model B2			150	250	
Model B3	two bays	100	200	200	50
Model B4			250	150	
Model B5			300	100	

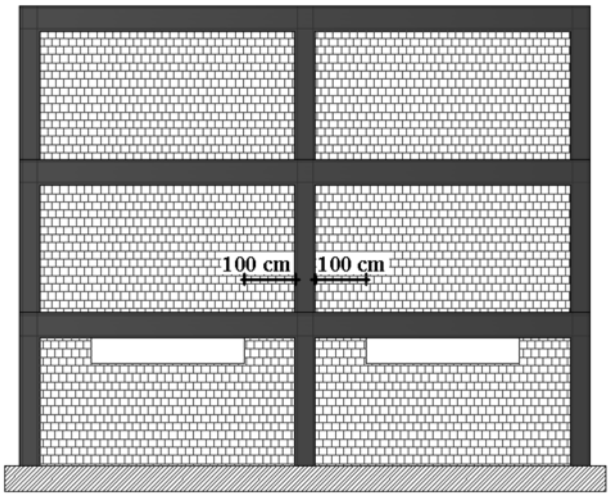


Fig. 14 The general form of window opening in model B buildings

opening. Window opening is located in two bays for Model B buildings (Fig. 14) while being present in one bay for Model A buildings. The wall distance between window opening and interior columns (L_{iw}) remains at the same value. The distance between window opening and exterior columns (L_{ew}) varied from 1 m to 3 m (Table 6).

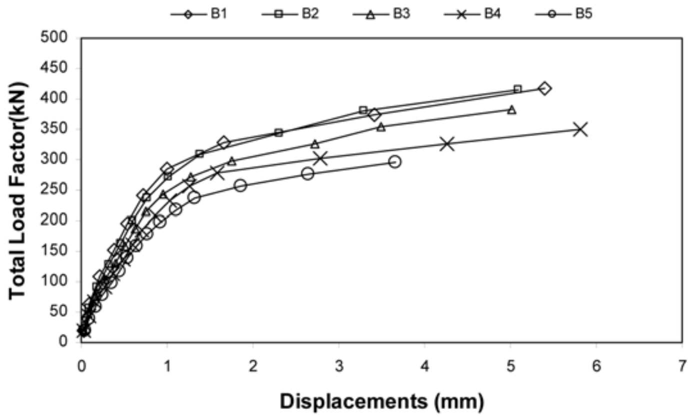


Fig. 15 The ground storey displacements of model B buildings

The all capacity of TLF values for the Model B buildings drops to about 250 kN as this value is 900 kN for the reference model. The capacity of TLF value is approximately 400 kN for the Model B1 and it is about 250 kN for the Model B5. As shown in Fig. 15, the capacity of TLF is about 70% lower than reference model. As the width of window opening (L_{iw}) increases, the capacity of TLF decreases. When the results of Model B and Model A are considered, decrease in stiffness for Model A buildings is lower than the one for with Model B buildings since the window opening is modelled for one bay.

5.3. Model C buildings

Window opening is located only in one bay while the other bay is filled with infill walls for Model C buildings (Fig. 16) as the Model A buildings. The difference between Model C and Model A is the distance between the interior column and window opening (L_{iw}) changing from 1000 to 500 mm. Depending on the change of L_{iw} value, the distance between the exterior column and window opening (L_{ew}) is varied from 0.50 m to 3.50 m (Table 7).

When the Model C buildings are compared with reference model in terms of the capacity of TLF, it is easily observed that the TLF values are dropt to around 350 kN. The capacity of TLF of buildings in which window opening is present, is about 60% lower than reference model as shown in Fig. 17.

Table 7 The forms of short columns of model C frame buildings

Models	Windows in	L_{iw}, (cm)	L_b, (cm)	L_{ew} (cm)	L_s (cm)
Model C1			100	350	
Model C2			200	250	
Model C3	one bay	50	250	200	50
Model C4			300	150	
Model C5			400	50	

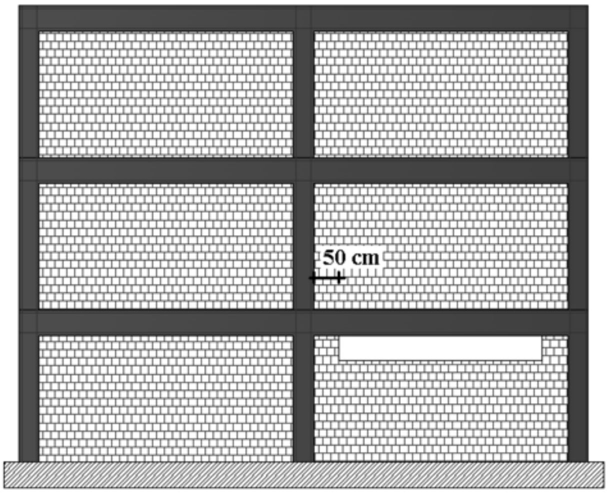


Fig. 16 The general form of window opening in Model C buildings

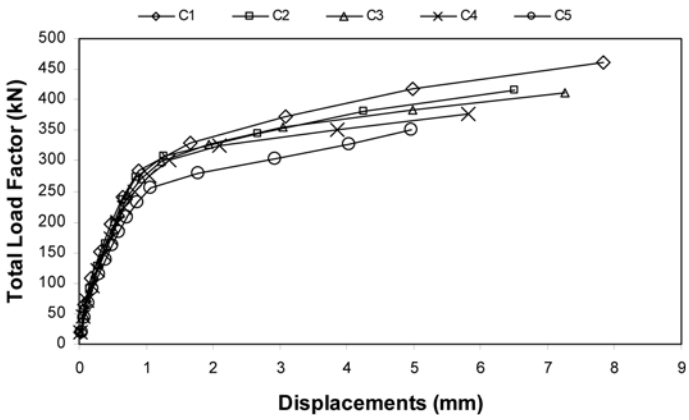


Fig. 17 The ground storey displacements of model C buildings

5.4. Model D buildings

Window opening is located in two bays for Model D buildings (Fig. 18) while it is located in one bay for Model C buildings. The only difference between Model D and Model C buildings is the existence of window opening. The parameters, which are used to describe the form of short columns, remained the same for both of main models. In model D buildings, the wall distance between window opening and interior columns is kept constant (L_{iw}=0.50 m) while the distance (L_{ew}) between window opening and exterior columns varied from 0.50 m to 3.50 m (Table 8).

The capacity of TLF values for the Model D buildings are about 250 kN as this value is 900 kN for the reference model. The capacity of TLF value is approximately 350 kN for the Model D1 and it is about 200 kN for the Model D5. As can be seen Fig. 19, the capacity of TLF is about 75% lower than the reference model. With increasing the length of window opening, the capacity of TLF in model D1 building is 50% lower than the one in Model D5 building.

Table 8 The forms of short columns of model D frame buildings

Models	Windows in	L_{iw}, (cm)	L_b, (cm)	L_{ew} (cm)	L_s (cm)
Model D1			100	350	
Model D2			200	250	
Model D3	two bays	50	250	200	50
Model D4			300	150	
Model D5			400	50	

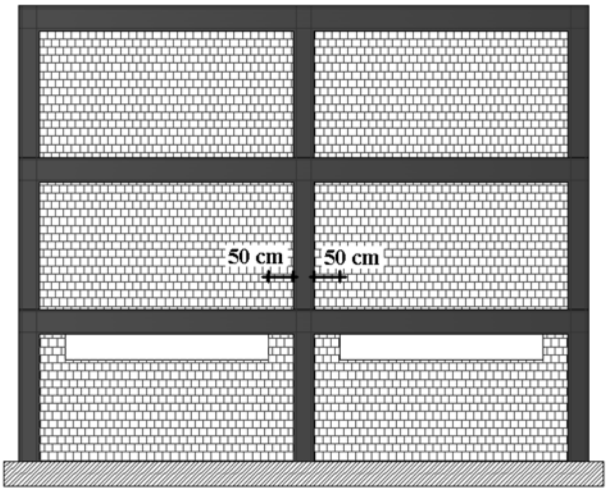


Fig. 18 The general form of window opening in model D buildings

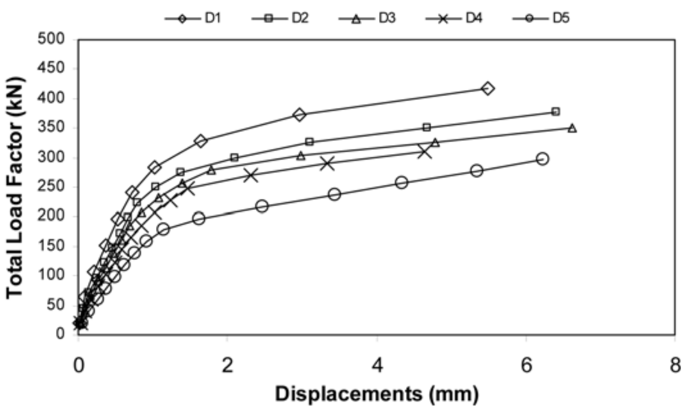


Fig. 19 The ground storey displacements of model D buildings

5.5. Model E buildings

For Model E buildings, window opening is located only in one bay while the other bay is filled with infill walls. The window openings are bonded without gap to interior column directly as can be seen in Fig. 20. Because window openings are bonded without gap to interior column directly, there

Table 9 The forms of short columns of model E frame buildings

Models	windows in	L_{iw}, (cm)	L_b, (cm)	L_{ew} (cm)	L_s (cm)
Model E1	one bay	0	100	400	50
Model E2			200	300	
Model E3			300	200	
Model E4			400	100	
Model E5			500	0	

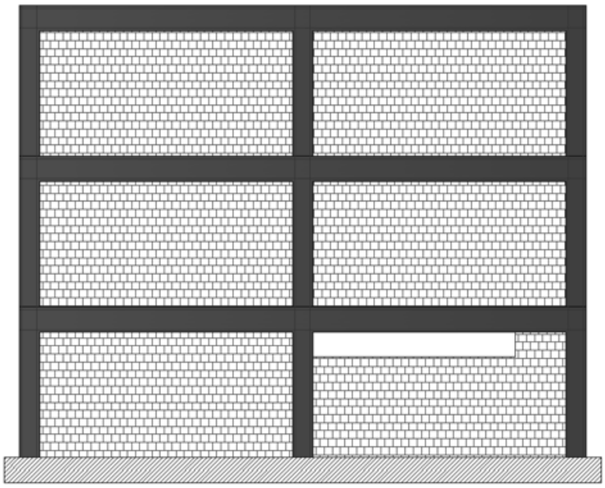


Fig. 20 The general form of window opening in model E buildings

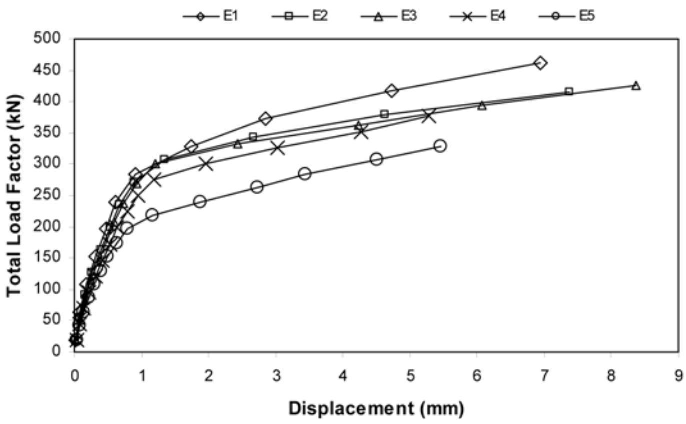


Fig. 21 The ground storey displacements of model E buildings

is no infill wall between window openings and interior column (L_{iw}=0) but infill wall is present between the window openings and the exterior column. The length of window opening is varied from 1 m to 5 m (Table 9). As the length of window opening increased up to length of bay in Model E5, no infill wall is left between the window openings and exterior column (L_{ew}=0).

The capacities of TLF for the Model E buildings drop to about 250 kN as this value is 900 kN for the reference model. The capacity of TLF value is approximately 400 kN for the Model E1 and it is about 250 kN for the Model E5. As shown in Fig. 21, the capacity of TLF is about 70% lower than that of the reference model. With increasing the length of window opening, the capacity of TLF is decreased. As a result of increasing the length of window opening, the capacity of TLF in model E1 building is 40% lower when compared with that of TLF in Model E5 building.

5.6. Model F buildings

The only difference between Model F and Model E buildings is the presence of a window opening. The parameters, which are used to describe the form of short columns, are the same for both of main models. Window opening is located in two bays for Model F buildings (Fig. 22) while it is located in one bay for Model E buildings. In model F buildings, there is no infill wall between window openings and interior column (L_{iw}=0) but infill wall is present between window openings and exterior column ($L_{ew} \neq 0$) like Model E buildings. The length of the window opening is varied from 1 m to 5 m (Table 10). As the length of window opening increased up to length of bay in Model F5, no infill wall is left between window openings and exterior column (L_{ew}=0).

In Model F buildings, the capacity of TLF which have long window opening is quite lower than the reference model as shown in Fig. 23. The capacity of TLF value is approximately 450 kN for the Model F1 and it is about 130 kN for the Model F5. With increasing the length of window

Table 10 The forms of short columns of model F frame buildings

Models	windows in	L_{iw}, (cm)	L_b, (cm)	L_{ew} (cm)	L_s (cm)
Model F1			100	400	
Model F2			200	300	
Model F3	two bays	0	300	200	50
Model F4			400	100	
Model F5			500	0	

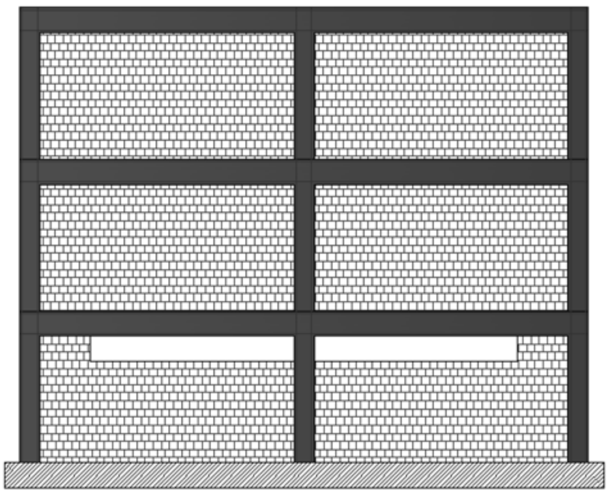


Fig. 22 The general form of window opening in model F buildings

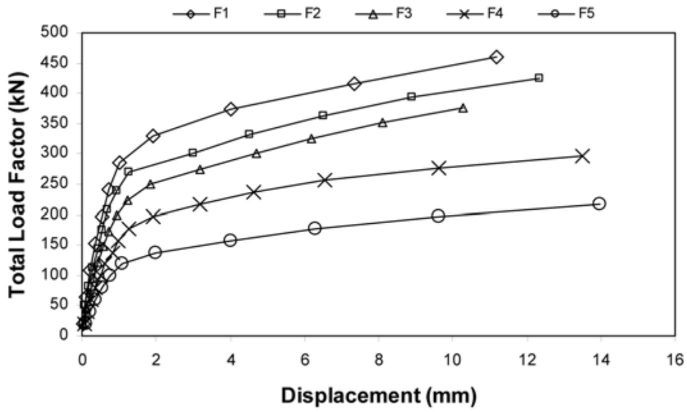


Fig. 23 The ground storey displacements of model F buildings

opening up to the length of bay, the capacity of TLF in model F1 building becomes 85% lower when it is compared with that of TLF in the reference model. When the results of Model F and Model E is considered, the decrease in stiffness for Model F buildings is higher when compared with that for Model E buildings since the window opening is modelled for two bays.

6. Conclusions

In this study the 31 different frame buildings containing short column problem are analyzed using a finite element method including material nonlinearities. Short column problem is generally developed due to the window openings which is put in infill walls between columns. Infill walls generally have positive effect to performance of buildings subjected to earthquake. However, it is clearly shown that the window opening located in infill wall due to architectural reason negatively affected the response of buildings in this study. If the window openings are adjoined to columns, its further negative effect to response of buildings is observed. In addition, this phenomena leads to formation of the short column. As can be seen from the results, if the window openings are constituted, the distance between the interior column and window opening should be at least 50 cm. It is also revealed that if the window openings are constructed along the bays, the capacity of TLF is decreased 85% compared with reference model in which all of bays are filled with infill walls.

The effect of the infill wall on the structure subject to earthquake is very important as shown in the study. The designer must take into account of the effect of the infill wall on the structure. During the design stage of new buildings, the short column effect should be avoided to the extent possible. If the avoidance of the short columns is not possible, then the requirements of the earthquake code must be satisfied.

Acknowledgements

We would like to thank Dr. Hafez Keypour for his support about LUSAS finite element program.

References

Arslan, M.H. and Korkmaz, H.H. (2007), "What is to be learned from damage and failure of reinforced concrete structures during recent earthquakes in Turkey?", *Eng. Fail. Anal.,* **14**, 1-22.

Dagangun, A. (2004), "Performance of reinforced concrete buildings during the May 1, 2003 Bingol Earthquake in Turkey", *Eng. Struct.,* **26**, 841-856.

Cagatay, I. (2005), "Failure of an industrial building during a recent earthquake in Turkey", *Eng. Fail. Anal.* **12**(4), 497-507.

Guevara, L.T. and García, L.E. (2005), "The captive- and short-column effects", *Earthq. Spectra,* **21**(1), 141-160.

Nikos, D., Lagaros, N.D. and Geraki, M.A. (2008), "The effects of construction practices on the seismic performance of RC frames with masonry infills", *Struct. Eng. Mech.,* **28**(1), 69-88.

Negro, P. and Verzeletti, G. (1996), "Effect of infills on the global behaviour of R/C frames: Energy considerations from pseudodynamic tests", *Earthq. Eng. Struct. D.*, **25**(8), 753-773.

Tegos, I. and Penelis, G.G. (1988), "Seismic resistance of short columns and coupling beams reinforced with inclined bars", *ACI Struct. J.*, **85**(1), 272-283.

LUSAS (2006), *Finite element system examples manual (version 13.6)*, FEA Ltd., U.K.

Erdik, M., Demircioglu, M., Beyen, K., Sesetyan, K., Aydinoglu, N., Gul, M., Siyahi, B., Onem, G., Tuzun, C., Salkon, A. and Kaya, Y. (2003), *May 01, 2003 Bingol (Turkey) Earthquake*, Department of Earthquake Engineering, Kandilli Observatory and Earthquake Research Institute, Bogazici University, Istanbul.

Ersoy, U. amd Ozcebe, G. (2001), *Reinforced concrete*, Evrim Yayınevi, Istanbul.

CC