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The effect of short columns on the performance of existing buildings

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Abstract. In this study, the seismic performance of a residential building which was damaged in the 1992 Erzincan (Turkey) Earthquake (Ms = 6.8) is performed. Damages on columns due to short columns are estimated analytically implementing the shear hinges and results are compared with the observed damages on the building after the earthquake. In seismic performance evaluation, a deformation based approach is adopted, whereby the structural behavior under external and seismic loads is evaluated. Furthermore, the effects of short columns formed by band windows in basement floors are investigated analytically. The sizes of band windows are parametrically changed in order to understand the effects of short columns on overall building behavior.

Keywords: short columns; seismic performance analysis; pushover analysis; non-linear time history analysis

1. Introduction

Post-earthquake damages on buildings provide vital information for the performance of structures under seismic loads. Detailed numerical modeling is necessary to define a level of performance after a dynamic load on structure. For these numerical models, properties which affect the overall behavior of structural systems remarkably, such as column-beam formation, sudden change in rigidity, band windows are important factors, which may be obtained throughout several suggested approaches located in the literature based on numerical simulations.

Worldwide infrastructures which are vulnerable to seismic lateral loads and located in high seismicity regions have stimulated the interest of many researchers to estimate the seismic damages (Koçak 2000, Koçak 2010, Koçak and Köksal 2010, Sucuoğlu et al. 2000, Elnashai 2000, UCTEA 2000, Ersoy 1993, Okutucu 1993). Damaged and undamaged buildings are thus examined and possible structural defects which may lead to the observed damages are determined. One of the most important irregularities that cause structural damage is the existence and the formation of short columns. Short columns, which are band windows on infilled or shear walls in basement floors, are considered, among others, within the Turkish Seismic Code of 2007 (TSC 2007). Also Işık and Pul (2006), Guevara and Garcia (2005), Çağatay (2007), Harumi et al. (1990), Kesti and Davies (1999), Arslan and Korkmaz (2007), examined short column effect on the overall behavior

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of buildings. Moretti and Tasios (2007) investigated the behavior of short columns under cyclic loads; Salau (2003), published his findings on long-term deformation of short columns. Doğan (2011), studied the effects of short columns formed by structural irregularities on buildings, Çağatay *et. al.* (2010), investigated the effect of short column of RC buildings and compare with the damages which occurred after Adana-Ceyhan earthquake. Bikce (2011), studied about a method which reduce short column effects in buildings with reinforced concrete infill walls on basement floors.

In Turkey, typically basement floors are utilized for various purposes. In order to illuminate the basement floor, band windows are constructed over the soil level, where the wall between columns is built up to a certain level and a gap is left for the window. Because of the rigidity of the wall below window level columns do not bend, so they are forced to bend within the short length of the window gap. The columns are exposed to massive shear forces under the effect of bending. There are many buildings in Turkey that have such structural faults.

Although most of the buildings in Turkey have band windows in the basement floor, they are generally ignored in the numerical model. When structural damages due to earthquake loads are critically examined, the effects of these gaps on the walls can clearly be seen Fig. 1. Gaps in the basement floor result in a negative overall outcome for the load-bearing system; thus it will be beneficial to avoid these gaps altogether.

In this study, the effects of short columns on the building performance are investigated for an existing building which was damaged in the 1992 Erzincan earthquake in Eastern Turkey. The seismic performance of the building is accomplished according to TSC (2007). The hinges occurred at various sections of the reinforced concrete (RC) building after the earthquake and the plastic hinges determined by the analysis are compared and a good agreement is observed. Furthermore, band windows are changed at various ratios to determine the seismic performance of buildings.

2. Examined building

Gulistan residential buildings built in Erzincan/Turkey is a three-storey residential building with a basement floor. Each storey has a height of 2.9 m and load-bearing system of the building consists of both RC frames and columns. The column dimensions are 0.25 m × 0.50 m, 0.30 m ×

Fig. 1 Short column damages

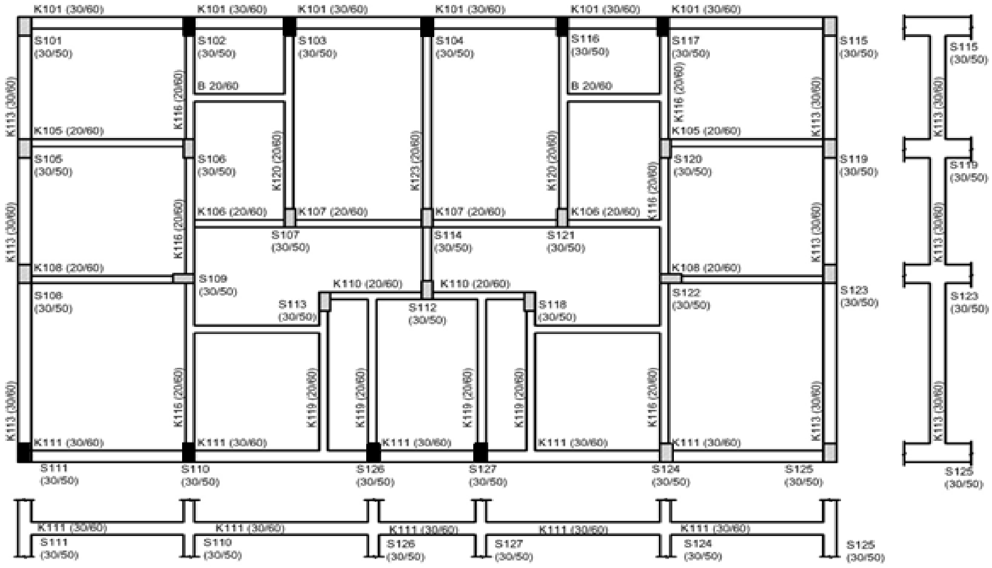


Fig. 2 Gulistan residential building plan

Fig 3 Gulistan residential building after the Erzincan Earthquake (1992)

0.50 m, and the beam dimensions are 0.25 m × 0.60 m. C14 class concrete (compressive strength is 14 MPa) is used with S220 (yield strength is 220 MPa) plain reinforcing steel. The effective ground acceleration coefficient of the building according to the TSC-2007 is $A_0 = 0.4$, importance factor of the building is I = 1, local site class is Z3 ($T_A = 0.15$ - $T_B = 0.4$s), and the live load participation factor n = 0.3. The plan of the Gulistan residential building is shown in Fig. 2

(Damaged columns due to earthquake are filled with black color). In the 1992 Erzincan Earthquake of magnitude 6.8, the building had shear damages on short columns (Fig. 3).

In this study, seismic demand and structural capacity curve according to TSC-2007 is used in analyses in order to calculate displacement demand at the top of the building, after which the system is pushed by this displacement until collapse. Then the band windows are reduced by 25%, 50%, and 75% in order to observe the change in the shear damage and collapse mechanism. For observe the shear damage mechanism, plastic hinges due to shear force are assigned to short columns around the band windows. The shear capacities of the short columns are calculated according to Turkish Reinforced Concrete Code (TS500-2000) by using equations below and presented in Table 1. Design shear force (V_r) (Eq.1) is calculated in TS500 as sum of cracking strength of concrete (V_{cr}) (Eq. 2) and contribution of transverse reinforcement (V_w) (Eq. 3). Also, upper limit of design shear force is calculated by using Eq. (4). In the equations below, f_{ctm} and f_{cd} are design strengths of concrete in tension and compression, b_w is width of the section, d is depth to steel bars, N_d is design axial load, A_c is section area, A_{sw} is total section area of transverse reinforcement, f_{ywd} is transverse reinforcement design yield strength, s is spacing of stirrups.

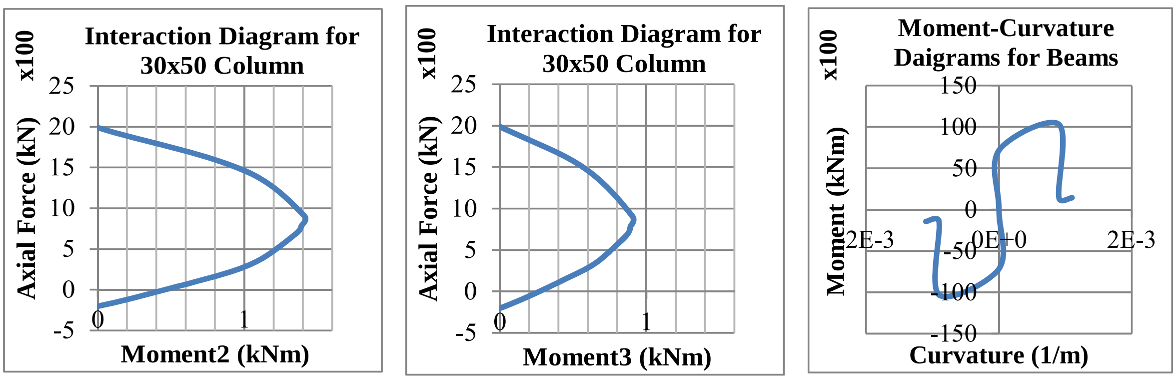
$$V_r = V_{cr} + V_w \tag{1}$$

$$V_{cr} = 0.65 \times f_{ctm} \times b_w \times d \times \left(1 + \gamma \times \frac{N_d}{A_c}\right) \tag{2}$$

$$V_w = \frac{A_{sw}}{s} \times f_{ywd} \times d \tag{3}$$

$$V_d \leq 0.22 \times f_{cd} \times b_w \times d \tag{4}$$

Shear capacities calculated by minimum value of Eq. (1) or Eq. (4) were entered to the SAP2000-ver.14.2 (2011) as a force controlled capacity shear hinge. Plastic hinges on due to bending hinges were deformation controlled hinges by using moment-curvature and P-M-M interaction diagram values of sections calculated by XTRACT-2007. Interaction and moment curvature diagrams of the column and beams are given in Fig. 4. No residual strength was considered after formation of hinge.

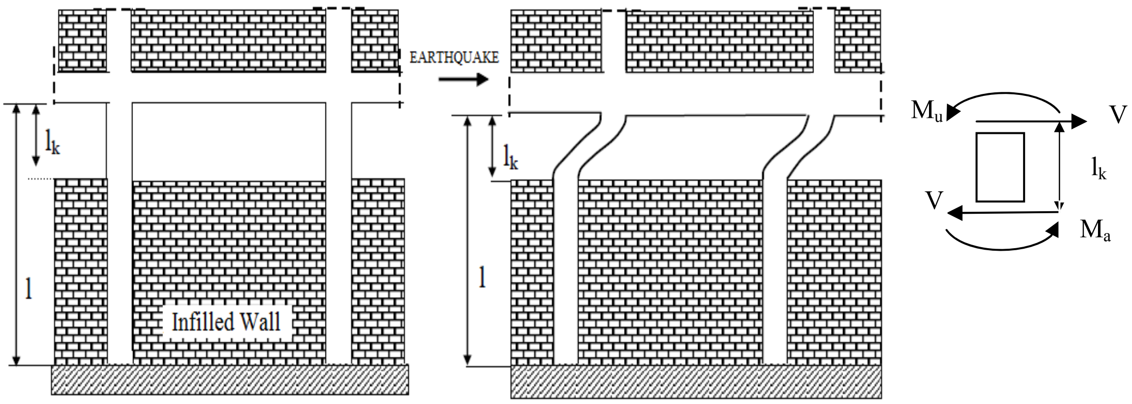


(a) (b) (c)

Fig. 4 (a), (b) Interaction diagrams for 30×50 column (c) Moment-curvature diagrams for beams

Table 1 Shear capacities of short columns

Column Number	bx cm	by cm	Binders	A_{sw} cm^2	s (distance) (cm)	f_{ctm} kN/cm^2	f_{ys} kN/cm^2	Vc=0.8×Vcr (kN)	Vs (kN)	Vr (kN)
external	30	50	ϕ8/10	1	10	0.140	220	92.82	55.00	147.82
internal	25	50	ϕ8/15	1	15	0.140	220	76.44	44.00	120.44



(a) (b) (c)

Fig. 5 (a) Construction frame with infill wall; b). Behavior of short column when horizontal load (e.g., earthquake) acts on structures; c). Internal loads effecting short column

3. Behavior of short columns

If the walls are short and connected to the frames, the main frame columns will not bend during an earthquake as a result of the horizontal forces and plane rigidity of the walls between two storeys. Columns are forced to bend as shown by the gap height on the upper section of the walls in Fig. 5 (a-c). Therefore high shear forces act on upper parts of the columns. Infill walls were modeled by the diagonal strut model suggested by Mainstone (1971).

From Fig. 5c, the shear force on short column (V) is expressed as the ratio of actual total bending moments on short column upside (M_u) and downside (M_a) for each loading step to column height (l_k) according to TSC (2007)

$$V = \frac{M_a + M_u}{l_k} \tag{5}$$

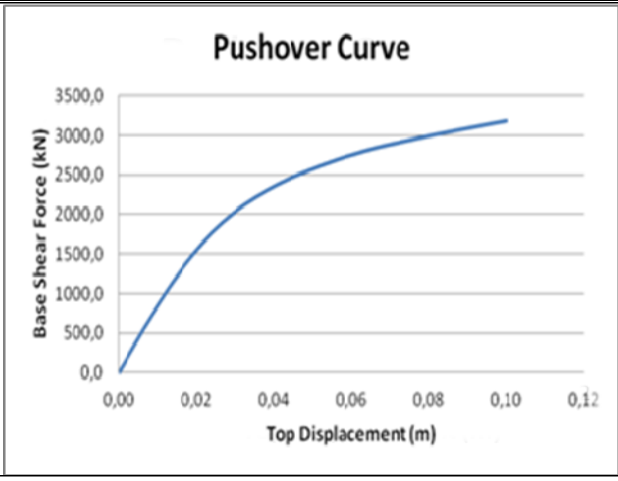
Here, M_a and M_u are the bending moments of the column which is unattached to the wall and l_k is the height of short column. Shear force on short column is the inverse ratio of short column height: the shorter that l_k becomes, the higher are the acting shear forces. Typically short column heights are 40-50 cm.

4. Evaluation of seismic performance

The inelastic non-linear behavior is utilized with respect to the materials; however, the lumped

Table 2 Pushover analysis, top displacement – base shear force corelation in X direction

Step	Displacement	Base Shear Force	Number of Hinges
n	$u_{x(m)}$	V (kN)	
0	0	0.0	0
1	0.008573	735.8	1
2	0.018902	1485.1	38
3	0.029602	2019.9	73
4	0.040988	2378.2	106
5	0.051898	2612.6	128
6	0.063415	2801.8	150
7	0.075794	2947.9	173
8	0.085964	3053.0	188
9	0.098412	3170.9	200
10	0.1	3185.9	202



plasticity model is taken as basis. In this model, which corresponds to the plastic support hypothesis in case of simple bending, it is assumed that the plastic deformations are evenly distributed all along the finite-length zones. The inner forces in the beam, column and frame-type load-bearing components are idealized as bar elements reaching their plastic capacities. In these models the plastic hinges placed on the end-zones of columns and beams, as well as shear hinges are defined for the basement columns. The basement walls are idealized as elastic plates and floor slabs are defined as rigid diaphragms. In order to show the actual behavior of concrete elements under seismic effects, mass participation rate and periods of the structure are calculated by using sectional rigidity for cracked sections. All calculations were made with SAP2000-ver.14.2 analysis program.

The obtained results are as follows: long direction period of the building is $T_{1x} = 0.546$s, mass participation rate is 70.1%, short direction period is $T_{1y} = 0.455$s, mass participation rate is 71%. As a result of the pushover analysis in the X direction, base shear force – top displacement values and base shear force – top displacement curve are shown in Table 2.

Modal capacity diagram is derived by using the pushover curve, and the modal displacement – modal acceleration plot as shown in Fig. 6(a). Superposing the modal capacity diagram with the behavior spectrum diagram gives the displacement demand of the building. In Fig. 6(b), spectral displacement value and the top displacement demand are obtained as

$$S_{d1} = C_{R1} S_{de1} = 1 \times 0.058 = 0.058 \text{ m} \tag{6}$$

$$u_{XN1} = \phi_{XN1} \Gamma_{X1} d_1 = 1 \times 1.278 \times 0.058 = 0.074 \text{ m} \tag{7}$$

The pushover analysis is repeated to determine the top displacement demand of the building. When the internal forces of elements are examined it is seen that short columns exceeded their shear capacity. Before the system completes its top displacement demand it will collapse as a result of shear fracture. In other words, the collapse mechanism of the system is brittle. In order to study this brittle failure, shear hinges are assinged to the short columns and the analysis is repeated. As a result of pushover analysis steps (9), shear hinges are formed in columns and the system loses its bearing capacity and fails in a brittle manner. In Table 3, values of top displacement – base shear force of columns with plastic hinges are given. First shear hinge occurred in 3rd step at about 735 kN base shear force. Also, columns with shear hinge on basement floor band windows after the ninth step (1794.559 kN base shear force) are marked.

As seen in Table 3, when top displacement reaches Δ = 2.49cm, shear fracture occurs on the columns (Fig. 7). While the top displacement demand of the structure during the design earthquake is $\Delta_{request}$ = 7.4cm, the system loses its bearing capacity at Δ = 2.49cm. These values show us that the building will be damaged; moreover, it is expected to collapse in an earthquake for which the probability of exceedance within a period of 50 years is 10%. Therefore, the Gulistan residential building was extensively damaged in 1992 Erzincan/Turkey Earthquake as a result of brittle failure of short columns on infill walls.

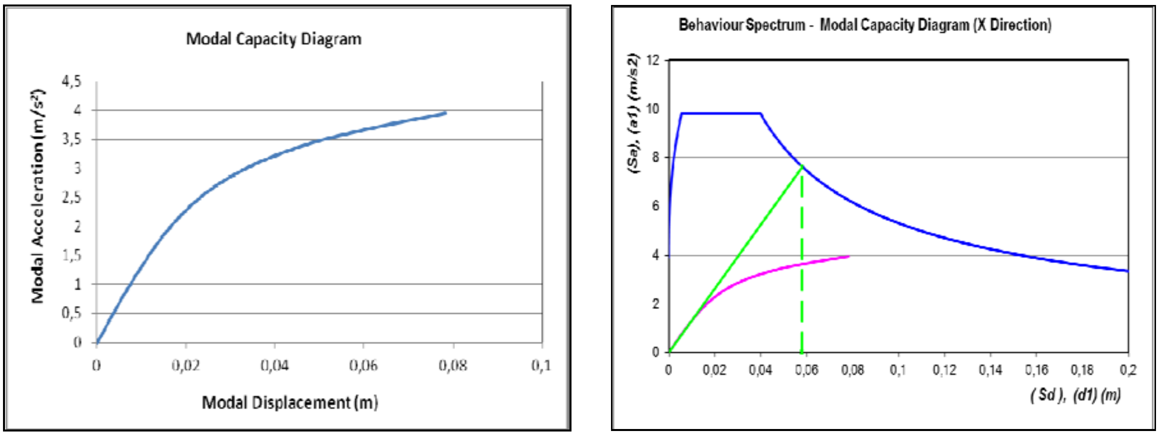
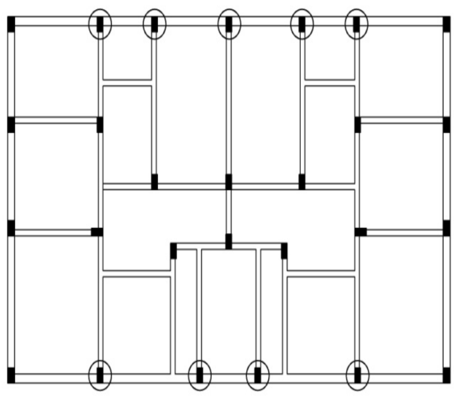
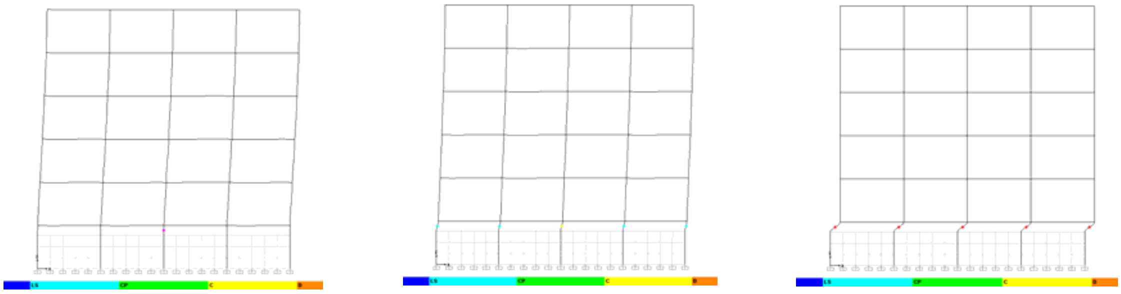


Fig. 6 (a) Modal capacity diagram (b) Superposing the behavior spectrum diagram with the modal capacity diagram

Table 3 Correlation between displacement and base shear force for shear hinge

Step	Displacement	Base Shear Force	Number of Hinges	Number of Shear Hinges
n	$u_{x(m)}$	V (kN)		
0	0.0002386	0	0	0
1	0.00286	245.458	0	0
2	0.00572	490.917	0	0
3	0,008573	735.769	1	0
4	0.011692	994.440	4	0
5	0.014798	1223.192	21	0
6	0.017668	1410.611	35	0
7	0.020626	1582.806	46	3
8	0.023601	1737.511	60	7
9	0.024878	1794.559	71	9





(a) ($u_{x(m)}$: 0.014798) (b) ($u_{x(m)}$: 0.023601) (c) ($u_{x(m)}$: 0.024878)

Fig. 7 Plastic hinges caused by shear fracture and corresponded top displacements on the building

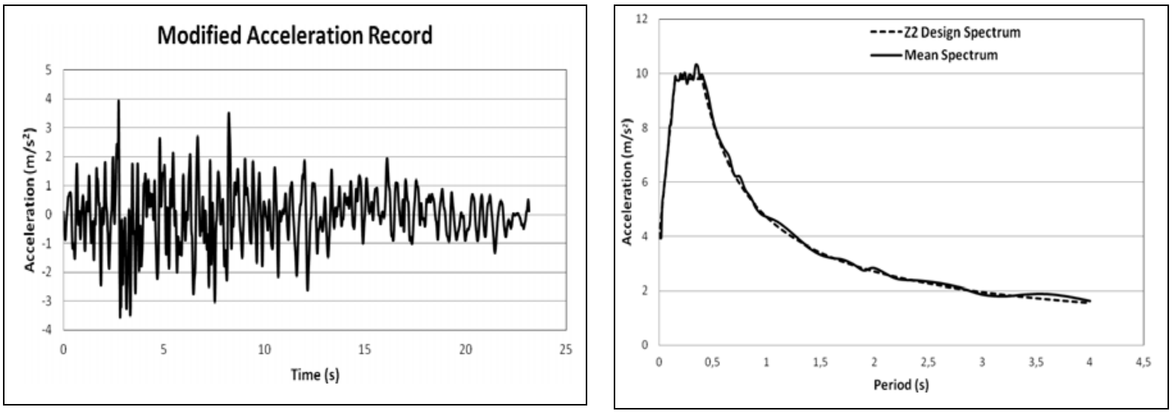


Fig. 8 Modified acceleration record and its comparison with TSCs design spectrum

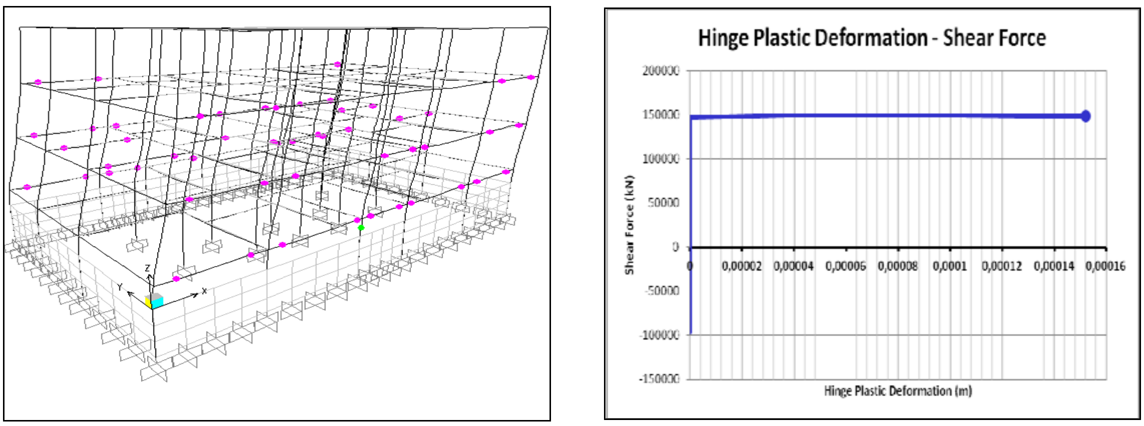


Fig. 9 The formation of shear hinge in the NTHA at time t = 0.61s

5. Non-linear time history analysis

As an alternative approach, the Non-linear Time History Analysis (NTHA) is used. This technique involves integrating the equation of movement of the system step-by-step using the

Table 4 Maximum and minimum fround shear forces between t = 0 and t = 0.61s for NTHA

OutputCase	CaseType	StepType	GlobalFX	GlobalFY	GlobalFZ	GlobalMX	GlobalMY	GlobalMZ
Text	Text	Text	kN	kN	kN	kN-m	kN-m	kN-m
NLTH	NonDirHist	Max	1262.541	0.998	11984.646	70721.158	-103456.9	9778.776
NLTH	NonDirHist	Min	-1512.707	-1.45	11950.286	70609.055	-122012.3	-7707.196

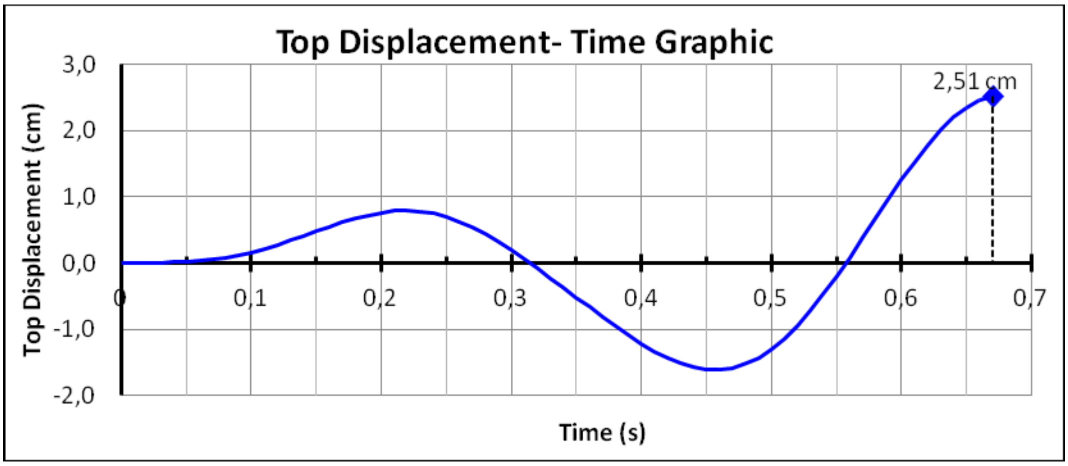


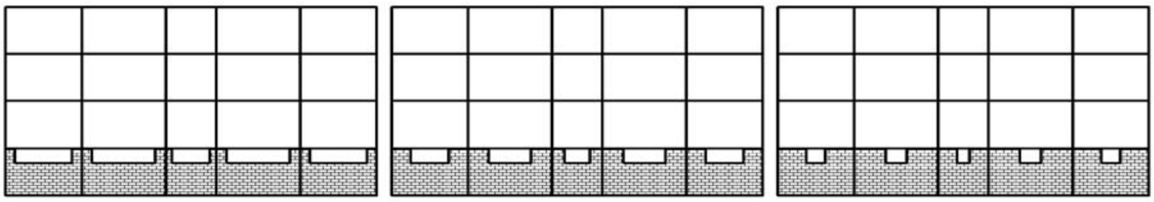
Fig. 10 Top point change in time

nonlinear behavior of the load bearing system. Displacement, plastic deformation and force that develop in the system at each step are calculated and the maximum values of those parameters corresponding to the seismic demand are determined. Based on these calculations, a modified acceleration record, which is compatible with TCS design spectrum, is obtained. The modified acceleration record and its mean spectrum curve's concurrency with TCSs spectrum curve are shown in Fig. 8.

As a result of the NTHA, shear hinges are formed in short columns at a time of t = 0.61s. This means that there is brittle fracture in the system. In Fig. 9, deformation of system at t = 0.61s is shown.

In Table 4, maximum and minimum ground shear force values are shown between t = 0 and t = 0.61s. In Fig. 10, the change of the top section within the same time zone is given.

The maximum ground shear force is V_t = 1512kN and the maximum top displacement is Δ = 2.51cm (Fig. 10). When these values are compared with the results of the pushover analysis, shear hinge on columns occurs when top displacement is Δ = 2.49cm and base shear force is 1795kN. These results are compatible with the results of NTHA. First shear hinge in NTHA occurred in both lower top displacement and base shear force due to pushover analysis. The reason of this should be load distribution of system. In fact, equivalent seismic load is considered as a triangular distribution in pushover analysis. It is also assumed that the equivalent seismic load is independent of the plastic section formations in the load bearing system. The structure could not withstand the top displacement at 7.4cm under design earthquake because of the brittle fractures on short columns, which is the reason for the heavy damages observed.



(a) (b) (c)

Fig. 11 (a) Band windows decreased by 25%; (b) Band windows decreased by 50%; (c) Band windows decreased by75%

Table 5 Decreasing band windows – change in top displacement

Decreased by 25%				Decreased by 50%				Decreased by 75%			
Step	Displacement	Base Shear Force	Number of Hinges	Step	Displacement	Base Shear Force	Number of Hinges	Step	Displacement	Base Shear Force	Number of Hinges
N	u_x(m)	V (kN)		N	u_x(m)	V (kN)		n	u_x(m)	V (kN)	
0	0	0.0	0	0	0	0.0	0	0	0	0.0	0
1	0.0074	642.8	0	1	0.0074	653.6	0	1	0.0074	663.4	0
2	0.008448	733.8	1	2	0.008295	732.6	1	2	0.008154	730.9	1
3	0.01593	1318.8	22	3	0.015696	1322.4	17	3	0.015684	1338.1	18
4	0.023488	1771.2	54	4	0.023547	1802.9	56	4	0.023178	1804.2	54
5	0.031012	2105.6	83	5	0.031174	2141.4	81	5	0.030898	2151.9	79
				6	0.038726	2389.6	91	6	0.039499	2433.7	91
				7	0.047104	2607.9	105	7	0.046975	2626.1	105
								8	0.054617	2771.0	122
								9	0.062424	2884.2	130
								10	0.071242	3000.2	140
								11	0.074	3032.6	143

6. Eliminating formation of short columns

The change in the behavior of buildings are examined by decreasing the band windows with brick walls around the columns. For this purpose, band windows are decreased by 25%, 50% and 75% (Fig. 11).

In Table 5, base shear force – top displacement values are presented with the decrease rate of the band windows.

In Table 5, when band windows are decreased by 25%, plastic hinges due to shear on short columns occur when top displacement reaches Δ = 3.1cm for the pushover analysis. With the decrease of band windows by 25%, top displacement increases from 2.49cm to 3.1cm. When band windows are decreased by 50%, the shear hinges on short columns occur when the top displacement value reaches 4.7cm. The system's top displacement capacity increases when the band windows are decreased. In the third step, band windows are decreased by 75%. In this case, system's top displacement reaches Δ = 7.4cm and there is no brittle fracture on any structural element. This system can sufficiently displace under design earthquake (Fig. 12). Fig. 13 shows the superpositioning of the behavior spectrum and modal capacity diagram. It can be concluded

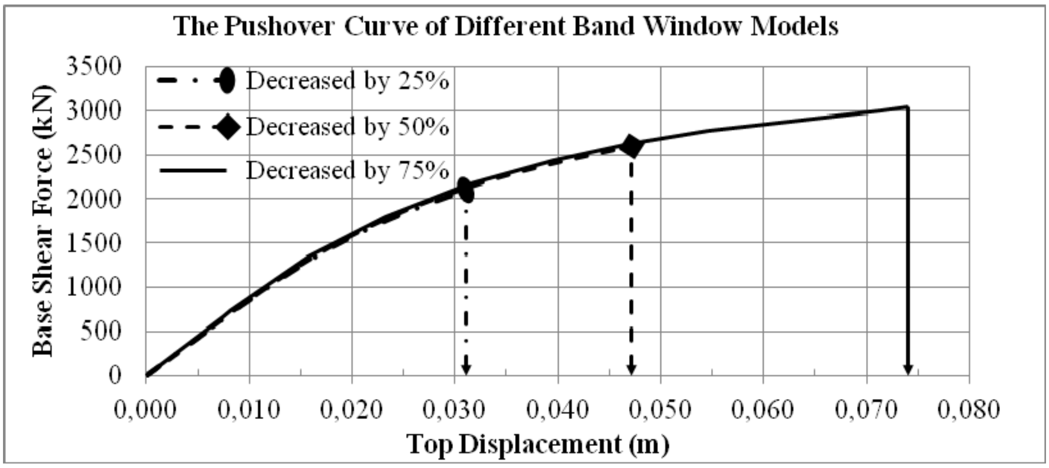


Fig. 12 The pushover curve of different band window models

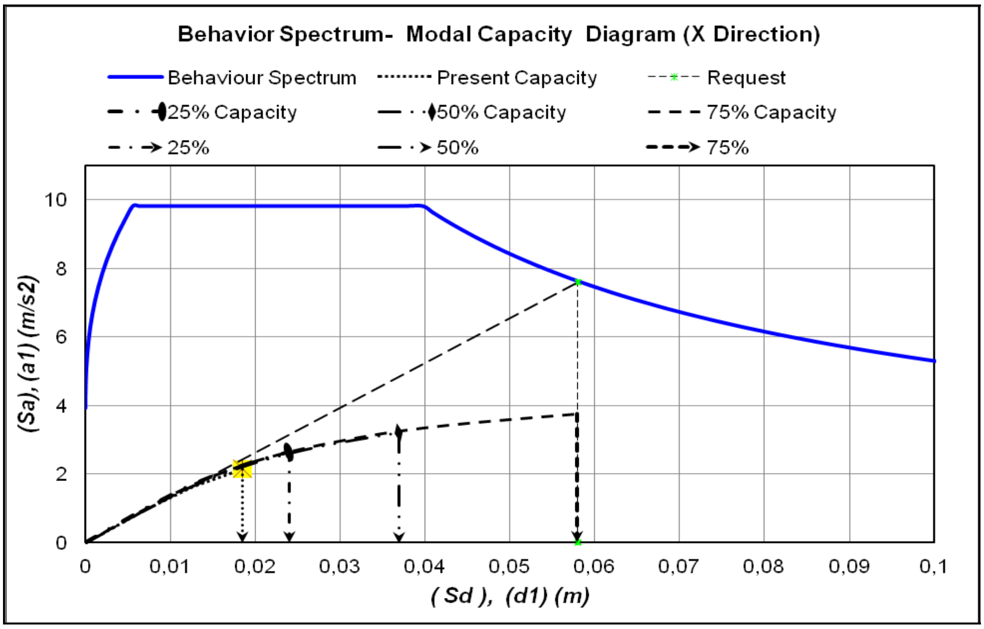


Fig. 13 Superposing of the behavior spectrum and modal capacity diagram

that if the band windows of the Gulistan building were 75% smaller, shear fractures would not have occurred and thus, the building would not have been massively damaged during the Erzincan Earthquake.

Shear hinges, which occurred at the last step of the pushover analysis, are marked with black dot in Fig. 14. Design shear capacity of short columns according to TS500 (2000) and Turkish Seismic Code of 2007, and values calculated by the models are given in Table 6.

It can be seen at the model which band windows decreased (Fig. 14(d)) 75%, there are no shear hinge occurred while shear damage occurred on the other models.

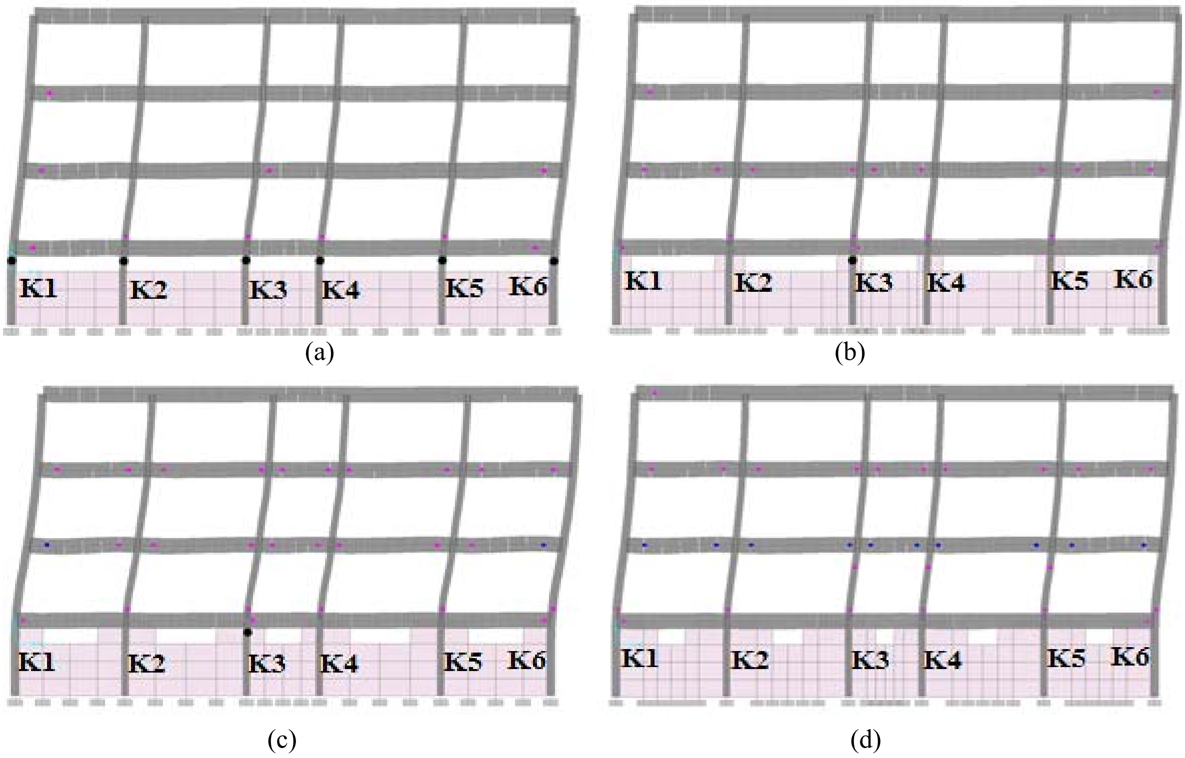


Fig. 14 Occurred shear hinges on the decreasing band windows (a) 0%, (b) 25%, (c) 50%, (d) 75%

Table 6 Design shear capacity of short columns according to TS500 (2000) and Turkish Seismic Code of 2007 and shear forces at short columns

	TS500 (kN)	TSC2007 (kN)	V (0%) (kN)	V (25%) (kN)	V (50%) (kN)	V (75%) (kN)
K1	147,82	185.892	79.47	53.46	49.89	43.32
K2	147,82	185.892	149.74	126.53	120.30	100.53
K3	147,82	185.892	**182.46**	**148.63**	**148.04**	142.43
K4	147,82	185.892	145.86	129.23	123.77	96.62
K5	147,82	185.892	163.71	145.93	137.57	118.19
K6	147,82	185.892	102.29	71.82	61.69	57.48

7. Conclusions

This paper aims to investigate short column effects on the overall building behavior. In this context, Gulistan building which was damaged in 1992 Erzincan/Turkiye Earthquake is examined. The following conclusions can be drawn based on the results of the analyses:

• The structural system could not sufficiently provide the demanded top displacement. Before the formation of bending hinge on short columns, brittle shear failure occurs.

• It is observed that shear forces in the push-over analysis reach the shear capacity of columns defined by TS500, that is, columns will suffer shear failure. Design rules of TS500 seems to be insufficient for proper design of short columns. Therefore, TSC 2007 must be used in the design process of short columns.

• Band window gaps have been decreased in three-dimensional numerical model, and gap effect on the behavior of building has also been examined. Firstly, the gaps on the left and right side of the short columns are decreased by 25%; shear fractures occur on the columns when the top displacement reaches 3.1cm according to pushover analysis. If the band windows are decreased by 50% shear fractures appear on the columns when the top displacement reached 4.7cm. Decreasing the length of the band windows resulted in positive outcome for the building. After decreasing the band windows by 75%, no shear fracture occurred in columns. Top displacement demand (7.4 cm) is provided by the system. The band windows from one column to another on the basement floor should have been made a quarter (¼) of their original dimensions to prevent the heavy damages and shear fractures on the columns in the Gulistan residential building.

• Short columns on multi-storey buildings, which are formed for architectural purposes, are critical design faults that can lead to destruction of building during a seismic event. It is clear that the shear strength calculated in accordance with the plastic moment capacity given in seismic codes is extremely important.

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