Earthquakes and Structures, *Vol. 15, No. 4 (2018) 383-393*
DOI: https://doi.org/10.12989/eas.2018.15.4.383

Seismic performance of low and medium-rise RC buildings with wide-beam and ribbed-slab

Kaan Turker[*1] and Ilhan Gungor[2a]

[1]*Balikesir University, Engineering Faculty, Department of Civil Engineering, Balikesir, Turkey*
[2]*Boga Engineering Company, Bursa, Turkey*

(Received September 25, 2017, Revised June 11, 2018, Accepted June 26, 2018)

Abstract. In this study, seismic performance of low and medium-rise RC buildings with wide-beam and ribbed-slab were evaluated numerically. Moment resisting systems consisting of moment and dual frame were selected as structural system of the buildings. Sufficiency of moment resisting wide-beam frames designed with high ductility requirements were evaluated. Upon necessity frames were stiffen with shear-walls. The buildings were designed in accordance with the Turkish Earthquake Code (TEC 2007) and were evaluated by using the strain-based nonlinear static method specified in TEC. Second order (P-delta) effects on the lateral load capacity of the buildings were also assessed in the study. The results indicated that the predicted seismic performances were achieved for the low-rise (4-story) building with the high ductility requirements. However, the moment resisting frame with high ductility was not adequate for the medium-rise building. Addition of sufficient amount of shear-walls to the system proved to be efficient way of providing the target performance of structure.

Keywords: RC building; wide-beam; moment resisting frame; dual frame; seismic performance evaluation

1. Introduction

Ribbed slabs are commonly preferred floor systems in buildings due to various architectural advantages. This floor system consists of a thin slab and ribs (shallow beams) supported on main beams. Spaces between the ribs are mostly filled with various materials (such as, hollow brick, autoclaved aerated concrete, styrofoam etc.) (Fig. 1). The use of ribbed-slab on buildings appeared primarily in Germany and surrounding countries where there are no earthquake effects. Afterwards, this floor system has become widespread in other countries experiencing severe earthquake disasters (such as, Italy, Spain, and Turkey).

The ribbed slabs are generally used with wide-beams having low height (equal to the rib height) in order to obtain a flat ceiling surface. Therefore, wide-beams are subjected to bending on the weak axis of beam. At the wide-beam joints, while some of the beam reinforcements pass through column core, the remaining pass outside the column core.

It is important to note, that majority of beam top and bottom reinforcements should pass from the column core in order to balance internal forces at joints (Paulay and Priestley 1992). For this reason, it is required that the beam width should not be very large compared to the column width. There are similar limitations regarding the beam widths in all of the modern design codes to balance the joint forces and sufficient development of bars (Table 1). The investigations on seismic behavior of wide beam-column subassemblies showed that aforementioned limitations are mostly adequate in terms of strength and deformation capacity of the joint (Gentry and Wight 1994, Quintero-Febres and Wight 2001).

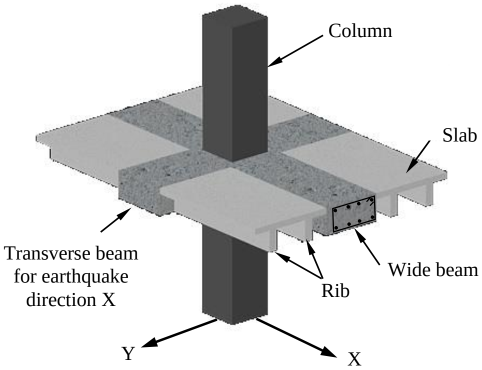
The other critical issue for the buildings with wide-beam is low lateral stiffness of the system compared to traditional systems. In order to avoid this disadvantage, it is required to use vertical members, such as shear-walls having higher lateral stiffness. Otherwise, high ductility demands in the members and second order issues can arise due to high lateral drifts. In the Turkish Earthquake Code (TEC 2007), when a structural system with wide-beam and ribbed-slab are selected, the level of earthquake excitation (seismic zone) and the level of member ductility become main criteria in seismic design of the building. For the buildings within moderate earthquake regions, the moment resisting frame system having the limited ductility can be used up to height of 13m. For taller buildings in this region, the dual frame systems such as moment frame and shear-wall have to be used in the design. However, the use high ductility structural systems is required for the buildings with-beam within high earthquake regions. It should be noted that the use of dual frame is not obligatory in a similar manner. In this regard, at high earthquake regions of Turkey, the use of moment resisting frames with high ductility is very widespread for low and medium-rise buildings with wide-beam and ribbed-slab. On the other hand severe damages and collapses were observed in the buildings with wide-beam during the past and recent earthquakes in Turkey (Malley *et al*.1993, Gulkan 1998, Cosar *et al*. 2011, Donmez 2013). The observations in especially new buildings have led to questioning the adequacy of design criteria as well as project and application errors in this type buildings.

∗Corresponding author, Assistant Professor
E-mail: kturker@balikesir.edu.tr
[a]MSc

Copyright © 2018 Techno-Press, Ltd.

ISSN: 2092-7614 (Print), 2092-7622 (Online)

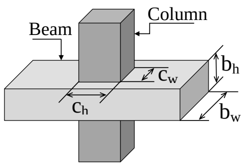
(a) Building structural system



(b) Typical joint detail

Fig. 1 An example of building with wide-beam and ribbed-slab

Table 1 The limitations in different design codes for wide-beams

	TS 500 (TS 2000)	$b_w \leq c_w + b_h$
	ACI 318-08 (ACI 2008)	$b_w < 3c_w$ $b_w < c_w + 1.5c_h$
	Eurocode-8 (CEN 2004)	$b_w \leq c_w + b_h$ $b_w \leq 2c_w$
	NZS3101-95 (NZSA 1995)	$b_w \leq c_w + c_h/2$ $b_w \leq 2c_w$

Seismic performance of beam-column joints in the RC buildings with wide-beams were investigated in various experimental studies on subassemblies (Gentry 1992, Stehle *et al*.2001, Benavent-Climent 2005, Kulkarni and Li 2008, Benavent-Climent *et al*. 2010, Goldsworthy and Abdouka 2011, Elsouri and Harajli 2013). However, the studies on seismic performance-based seismic evaluation of these structural systems are quite limited.

Donmez (2013), summarized the observations of buildings with wide-beam and ribbed-slab after the Turkey earthquakes and performed numerical studies. The numerical results of study showed that displacement demands of the considered buildings were nearly 40% more than those of both low and medium-rise traditional buildings. The study also showed that the predicted performance target was not achieved for a RC frame building designed in accordance with the TEC (2007). Almansa *et al*. (2013) investigated the seismic performances of buildings with wide-beams, which were not designed sufficiently with respect to seismic details in moderate earthquake region of Spain. In the study, low and medium-rise buildings were numerically investigated by nonlinear static and dynamic methods as well as the code-based method. Effect of infill walls and its contribution on seismic performance were also considered. The study results showed that the seismic performances of this type of buildings were insufficient in general manner. In addition, the infill walls have significant effect on the seismic performance depending on the infill ratio.

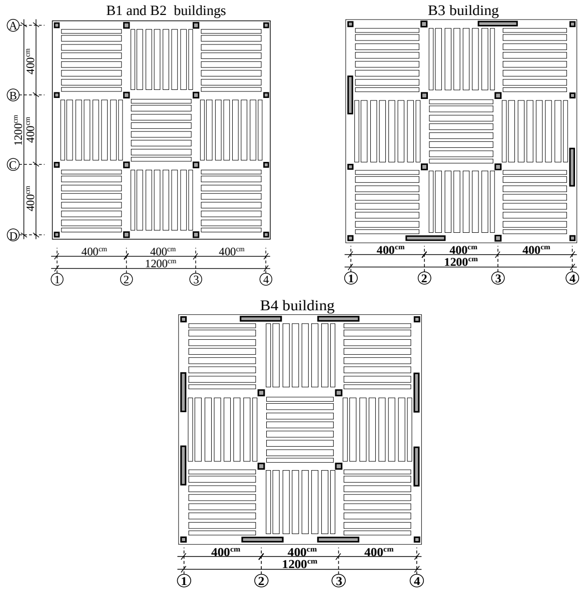


Fig. 2 Typical floor plans of the buildings

In this study, seismic performances of four different RC buildings with the wide-beam and ribbed-slab, which represent low and medium-rise buildings and designed in accordance with the Turkish Earthquake Code (TEC 2007), were evaluated for the moderate and severe earthquake levels. The structural systems of buildings consist of the moment resisting frame and dual frame (moment frame and shear-wall). Sufficiency of the moment resisting frames designed with high ductility and necessity of the shear-walls (dual frame) were discussed. The performance evaluations were carried out by the strain-based nonlinear static analysis method specified in the TEC for two earthquake levels (severe and moderate earthquake). Second-order (p-delta) effects on the lateral load capacity of buildings were also assessed in the study.

2. Analyzed buildings

Four different building models including the wide-beam and ribbed-slab were used for the performance evaluations. Typical floor plans and elevations of the buildings are shown in Figs. 2 and 3. 4-story and 7-story buildings were selected to represent the low and medium-rise buildings, respectively. The structural systems of buildings B1 and B2 consist of the moment resisting frames and the buildings B3 and B4 are the dual frames. While there is only one shear-wall in outer frame of the building B3, the shear-wall ratio for the B4 is doubled (Figs. 2 and 3).

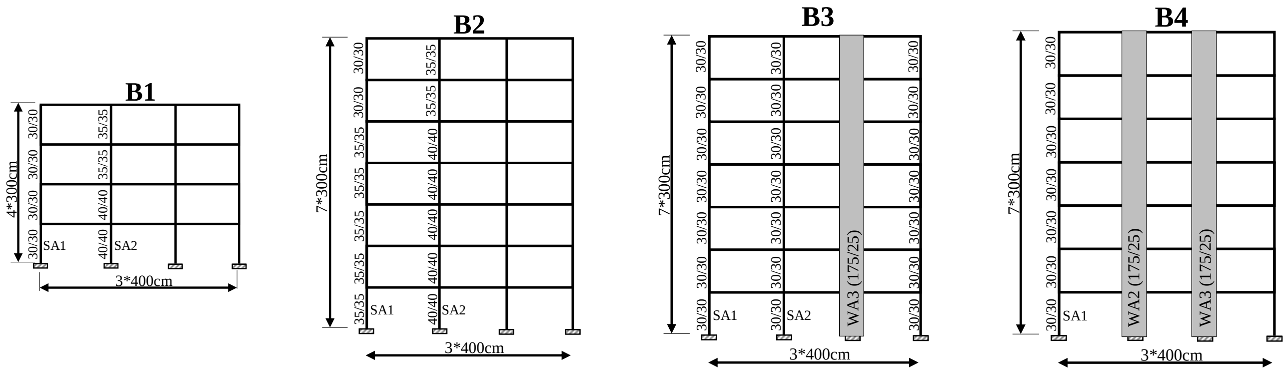


Fig. 3 Elevations of the buildings (for axis A)

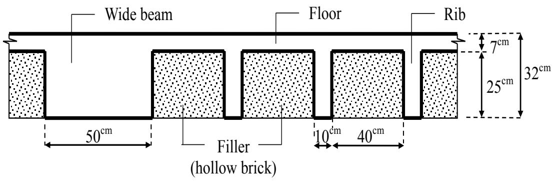
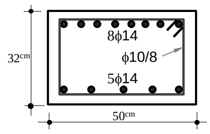


Fig. 4 Typical ribbed-slab section

Table 2 Beam reinforcement details for the buildings B1 and B2

Building B1	Beam reinforcements			Building B2	Beam reinforcements		
Story	Top/bottom	Left end	Right end	Story	Top/bottom	Left end	Right end
1	Top	8ϕ14	8ϕ14	1	Top	9ϕ14	9ϕ14
	Bottom	5ϕ14	5ϕ14		Bottom	5ϕ14	5ϕ14
2	Top	7ϕ14	7ϕ14	2-3	Top	9ϕ14	9ϕ14
	Bottom	4ϕ14	4ϕ14		Bottom	6ϕ14	6ϕ14
3	Top	6ϕ14	6ϕ14	4	Top	9ϕ14	9ϕ14
	Bottom	3ϕ14	3ϕ14		Bottom	5ϕ14	5ϕ14
4	Top	4ϕ14	4ϕ14	5	Top	7ϕ14	7ϕ14
	Bottom	3ϕ14	3ϕ14		Bottom	4ϕ14	4ϕ14
Typical beam detail (Story one)	 8ϕ14, ϕ10/8, 5ϕ14, 32cm, 50cm			6	Top	6ϕ14	6ϕ14
					Bottom	4ϕ14	4ϕ14
				7	Top	4ϕ14	4ϕ14
					Bottom	4ϕ14	4ϕ14

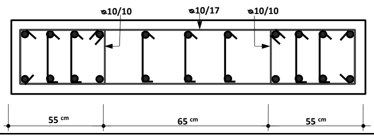
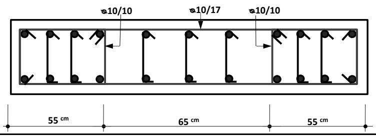
The buildings were designed to provide the high ductility requirements of the Turkish Earthquake Code (TEC 2007) for the severe earthquake level with 10% probability of exceedance in 50 years. The concrete strength, reinforcement yield strength and reinforcement tensile strength were taken constant as 25 MPa, 420 MPa and 550 MPa, respectively. Beams were designed as the wide-beam with the dimension of 50 cm×32 cm (Fig. 4). The beam height was selected to be compatible with the ribs and beam width meets the code criteria given in Table 1. The beam reinforcement details for the buildings B1 and B2 are given in Table 2. In order to reveal the most unfavorable seismic performance of wide-beam buildings, the minimum cross-sectional dimensions satisfying the code (TEC 2007, TS 2000) requirements were used in the design of the columns. The column and shear wall cross-sectional dimensions and reinforcements for the first story of buildings are given in Table 3. The details of columns in other stories are similar. However, dimensions of some columns have been reduced at upper stories (Fig. 3). Detailed information regarding the columns and beams of buildings can be found in Gungor (2014). The dead load (G) of 1.4 kN/m^2 and the live load (Q) of 2.0 kN/m^2 were considered at each floor of buildings. In addition, infill loads of 8.2 kN/m were considered on all wide-beams.

3. Numerical modeling of the analyzed buildings

The nonlinear static analysis method (Incremental equivalent earthquake load method) specified in the Turkish Earthquake Code (TEC 2007) was used for the performance evaluations. In the nonlinear analyses, the lumped plasticity (plastic hinge) approach was accepted and the second-order (p-delta) effects were considered. The combination of G+ 0.3Q was considered for the constant gravity loading of the buildings. The lateral load patterns compatible with the first mode shape of the buildings were used during the nonlinear analyses. The gross flexural stiffness of reinforced concrete members were reduced by the coefficients recommended in TEC

Three-dimensional mathematical models were prepared for the nonlinear analyses. All member ends were modeled with hinges showing the bilinear rigid-plastic behavior. For this purpose, the moment-curvature analyses were conducted for the critical sections of member. The moment-curvature relations of column members were obtained for constant axial force considering the vertical loadings. However, the moment-axial force interaction diagrams were used in formation of the plastic hinges. Damage levels of the members were determined based on material (reinforcement and cover/core concrete) strains unlike the other performance-based codes (Eurocode-8 2008, ASCE/SEI41-13 2013 etc.). Material based strain demands were obtained by use of the moment-curvature analyses and the plastic rotation demands of members. Three damage limits and four damage ranges are defined for ductile members in the TEC as shown in Fig. 5. The member damage limits based on the material strains are given in Table 4 (TEC 2007). According to the TEC, the most critical material strain over the reinforcement or concrete

Table 3 Section details of columns and shear-walls for buildings' first stories

Building	Member	Sections (cm/cm)	Longitudinal reinforcement	Transverse reinforcement	Typical transverse reinforcement details
B1	SA1	30/30	4ϕ18	ϕ10/10	SA1 / SA2 and SB2 
	SA2	40/40	8ϕ16	ϕ8/10	
	SB1	40/40	8ϕ16	ϕ8/10	
	SB2	35/35	8ϕ14	ϕ10/10	
B2	SA1	35/35	8ϕ14	ϕ10/10	SA1 / SA2 and SB2 
	SA2	40/40	8ϕ16	ϕ10/10	
	SB1	40/40	8ϕ16	ϕ10/10	
	SB2	40/40	8ϕ16	ϕ10/10	
B3	SA1	30/30	4ϕ18	ϕ12/10	WB1 
	SA2	30/30	4ϕ24	ϕ12/10	
	WB1	25/175	22ϕ22	ϕ10/10	
	SB2	40/40	8ϕ16	ϕ10/12	
B4	SA1	30/30	4ϕ18	ϕ12/10	WA1 
	WA1	175/25	16ϕ20-6ϕ14	ϕ10/10	
	WB1	25/175	16ϕ20-6ϕ14	ϕ10/10	
	SB2	40/40	8ϕ16	ϕ10/12	

S: column, W: shear wall, Other letters and numbers denote the name of axis

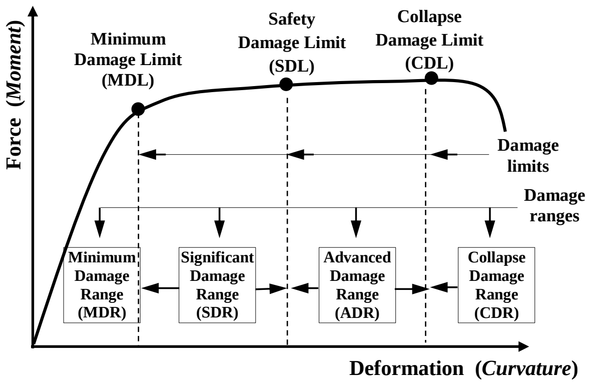


Fig. 5 The definitions of damage limits and ranges in TEC

Table 4 The used strain limits for damage limits

Strain	Minimum Damage Limit (MDL)	Safety Damage Limit (SDL)	Collapse Damage Limit (CDL)
Cover concrete strain (ε_{cc})	0.0035	---	---
Core concrete strain (ε_{co})	---	0.0135	0.0180
Reinforcement strain (ε_s)	0.0100	0.0400	0.0600

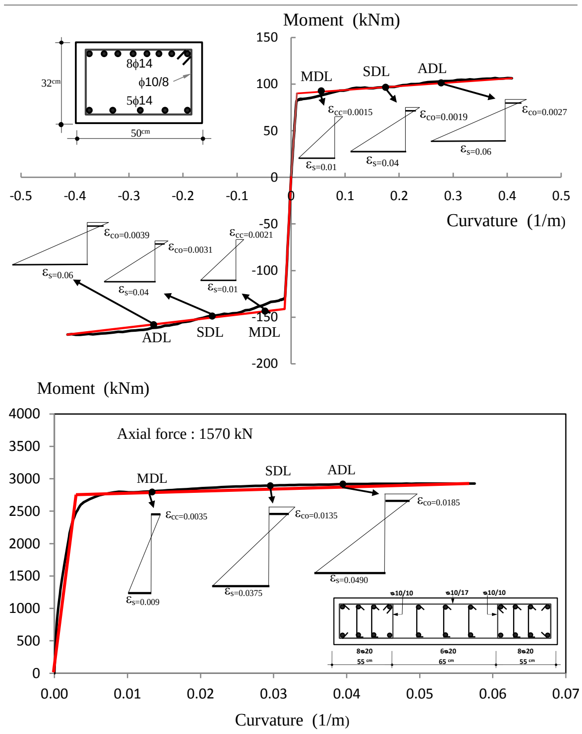


Fig. 6 Moment-curvature relations and damage limits of a beam and shear-wall (from building B1 and B3)

are used in evaluation of the member damage. The unconfined cover concrete strain (ε_{cc}) is only used in the determination of the "minimum damage limit". The confined core concrete strain (ε_{co}) is used for the further damage limits. The moment-curvature relation and the damage limits for a beam and a shear-wall member are shown in Fig. 6. SAP2000 (2008) and XTRACT (2004) software were used for the structural and section analysis, respectively.

4. Results of the performance-based analysis of the buildings

The performance evaluations of the wide-beam buildings were investigated for severe (design) earthquake (E1) and moderate earthquake (E2) levels. The probabilities of exceedance in 50 years for the E1 and E2 are 10% and 50%, respectively. The E2 earthquake is

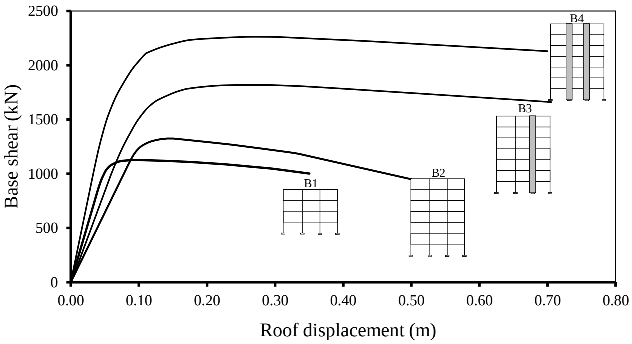


Fig. 7 Capacity curves of the buildings

Table 5 First modal properties and earthquake demands of the buildings

Building		B1	B2	B3	B4
Seismic weight (kN)		7914	14328	14052	14095
Period (T_1) (s)		1.016	1.619	1.349	1.016
Mass part. ratio (%)		82.4	80.2	75.3	75.9
Modal part. fact.(Γ_{x1})		76.7	101.8	106.8	105.4
Max. modal aml. (Φ_{x1})		0.017	0.013	0.013	0.013
Modal disp. demand (m)	E1 eart.	0.117	0.21	0.175	0.124
	E2 eart.	0.056	0.109	0.088	0.062
Roof disp. demand (m)	E1 eart.	0.149	0.285	0.237	0.166
	E2 eart.	0.071	0.143	0.119	0.083
Base shear demand (kN)	E1 eart.	1115	1230	1807	2230
	E2 eart.	1117	1330	1640	1900

defined as the half of the E1 earthquake spectral acceleration values.

In the Turkish Earthquake Code (TEC 2007) as well as in many other seismic codes, seismic design of a building aims limitation of the plastic deformations in order to ensure life safety at the design earthquake (E1). However, it is also expected that the damages occurred in the moderate earthquakes (E2) remains at a repairable level although no calculation is made during the design phase. In this study, damaged based performance levels specified in the TEC are used for assessment of the designed buildings. Three building performance levels are defined in the TEC: “Immediate Occupancy”, “Life Safety” and “Pre-Collapse”. The Life Safety performance level should be ensured for the earthquake E1 controlling the damage index without partially or total collapse. The repairable damages is stipulated for the earthquake E2 and achievement of the Immediate Occupancy performance level representing the lowest damage state is expected, as in the scope of this study.

Firstly, the capacity curves (roof displacement-base shear relation) of the buildings were obtained from the non-linear static analyses (Fig. 7). Later, the capacity curves were transformed into modal capacity diagrams by means of Eqs. (1) and (2) and the first modal properties of buildings, as shown in Figs. 8-10.

$$a_i = \frac{V_i}{M_1} \tag{1}$$

$$d_i = \frac{\delta_i}{\Phi_1 \Gamma_1} \tag{2}$$

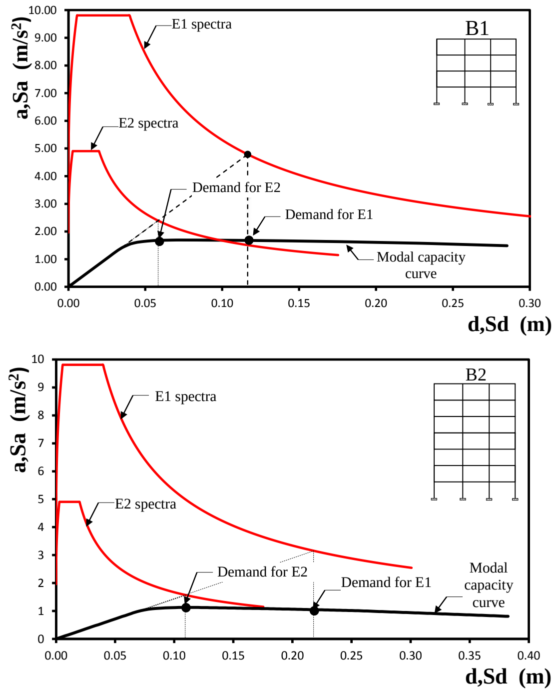


Fig. 8 Modal capacity curves and displacement demands for the buildings B1 and B2

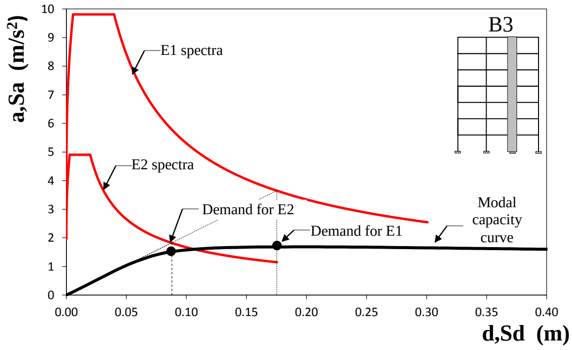


Fig. 9 Modal capacity curves and displacement demands for the building B3

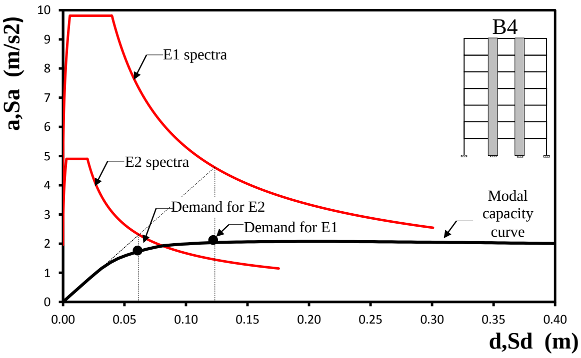


Fig. 10 Modal capacity curves and displacement demands for the building B4

Here, a_i is the modal acceleration at the (i) th step of nonlinear analysis, V_i is the base shear force obtained at the end of the (i) th step, M_1 is the effective mass related to the first mode, d_i is the modal displacement, δ_i is the roof

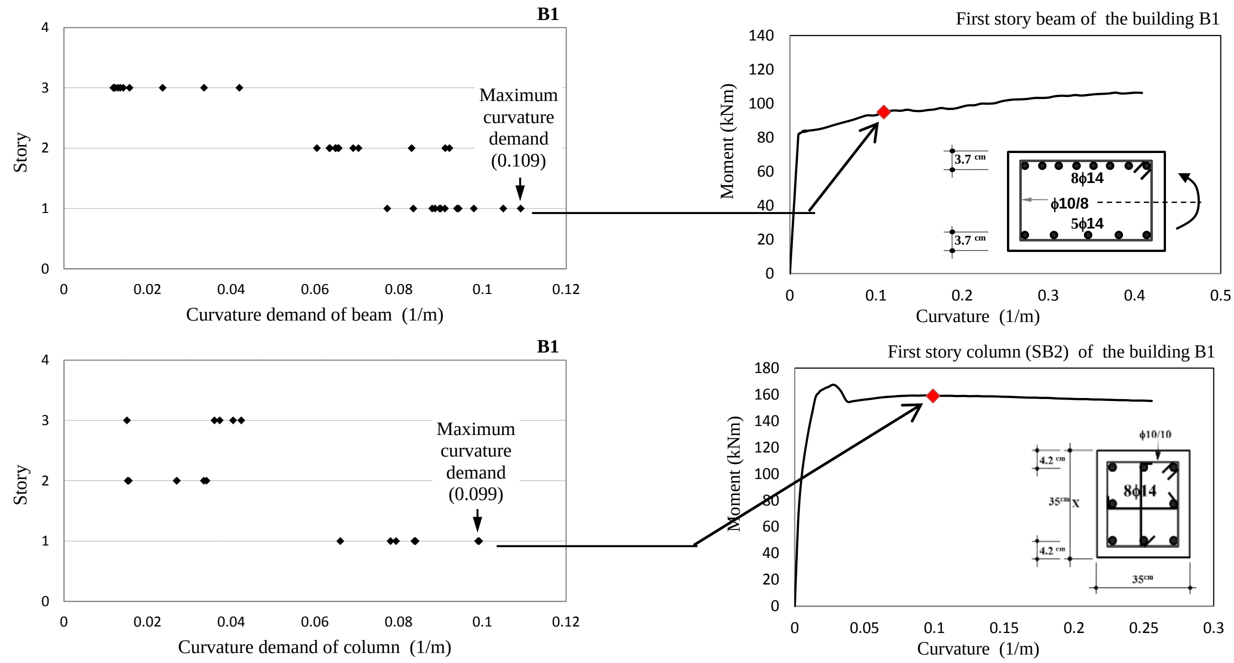


Fig. 11 Curvature demands of the building B1 for the earthquake E1

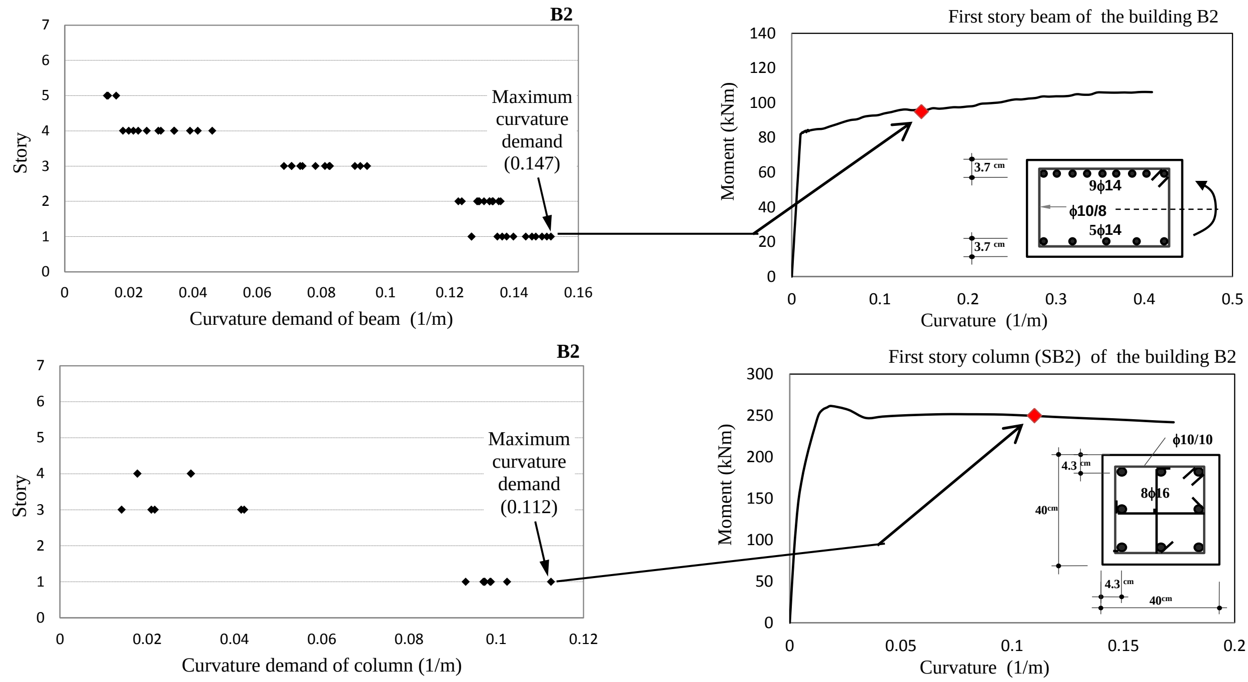


Fig. 12 Curvature demands of the building B2 for the earthquake E1

displacement of the system obtained at the end of the (i) th step, Γ_1 is the modal participating factor of the first mode in the corresponding earthquake direction, Φ_1 is the maximum (roof) amplitude of the first mode.

The first mode properties calculated for the buildings are summarized in Table 5. As shown in Figs. 8-10, the modal displacement demands were determined by using the acceleration spectra related to the earthquakes E1 and E2. Finally, the modal displacement demands were converted to the roof displacement demands with an inverse operation (Table 5).

The selected structural systems were pushed to the displacement demands for the two defined earthquake levels. Plastic rotation demands (θ_p) were obtained for all the critical sections (member ends). The plastic and total curvature demands (χ_p and χ_T) were calculated by Eqs. (3) and (4), respectively.

$$\chi_p = \theta_p / L_p \tag{3}$$

$$\chi_T = \chi_y + \chi_p \tag{4}$$

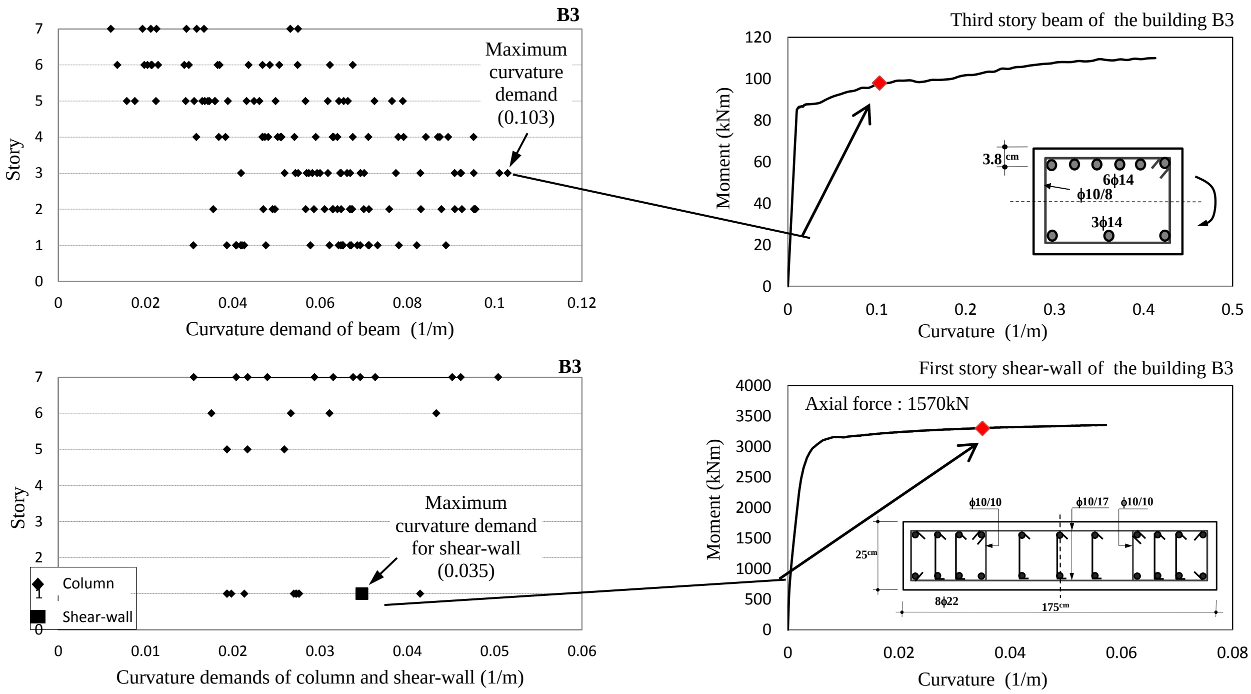


Fig. 13 Curvature demands of the building B3 for the earthquake E1

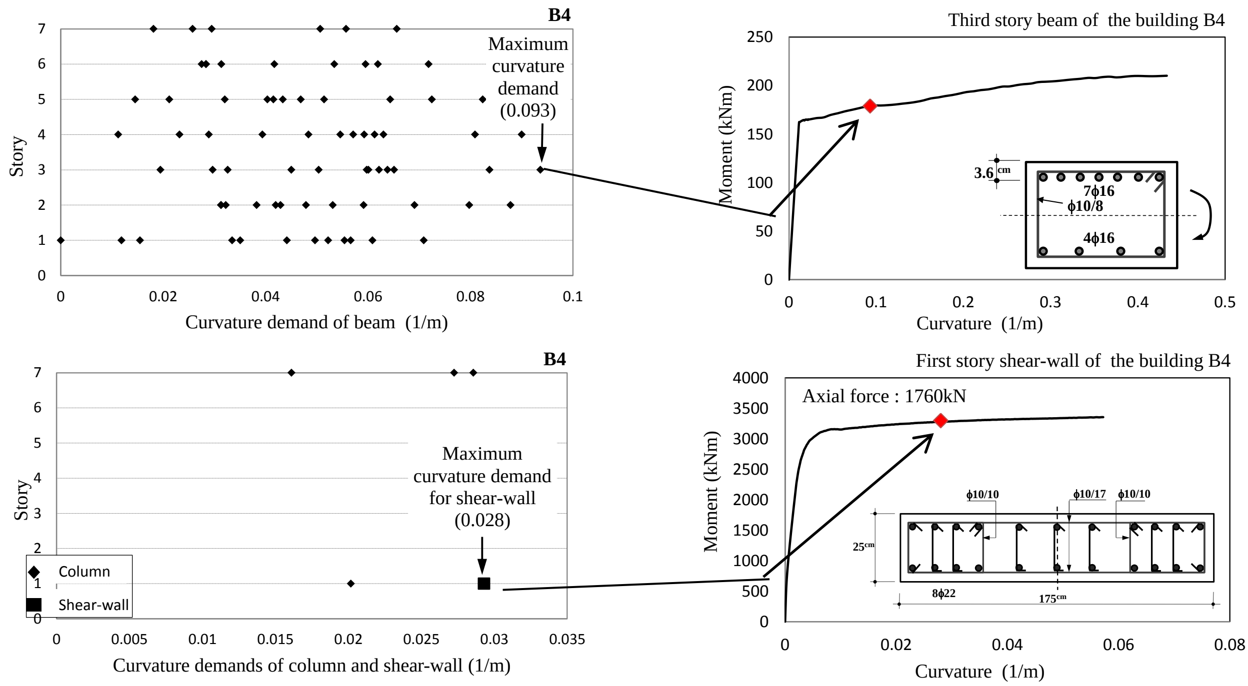


Fig. 14 Curvature demands of the building B4 for the earthquake E1

Here L_p and χ_y defines the plastic hinge length and the yield curvature of section, respectively. L_p was assumed as half the height of the section in accordance with the TEC. The yield curvature (χ_y) were determined from the moment-curvature analysis. The calculated member curvature demands are shown in Figs. 11-14 for the yielded sections under the earthquake E1. In addition, the curvature demands causing the maximum damage in member are shown on the moment-curvature graphic of the related member (Figs. 11-14). The material strain demands (ε_{cc}, ε_{co} and ε_s) for members were determined based on the total curvature demands. The damage ranges (MDR, SDR, ADR, CDR) of members were obtained comparing these demands with the strain limits specified in the TEC (Table 4).

The damage ranges obtained for the E1 earthquake are shown on the structural systems by using scaled circles (Fig. 15). However, although the non-yielding (elastic) sections were also accepted as to be in the minimum damage range (MDR) according to the TEC, only the sections that passed the yield point are marked in order to

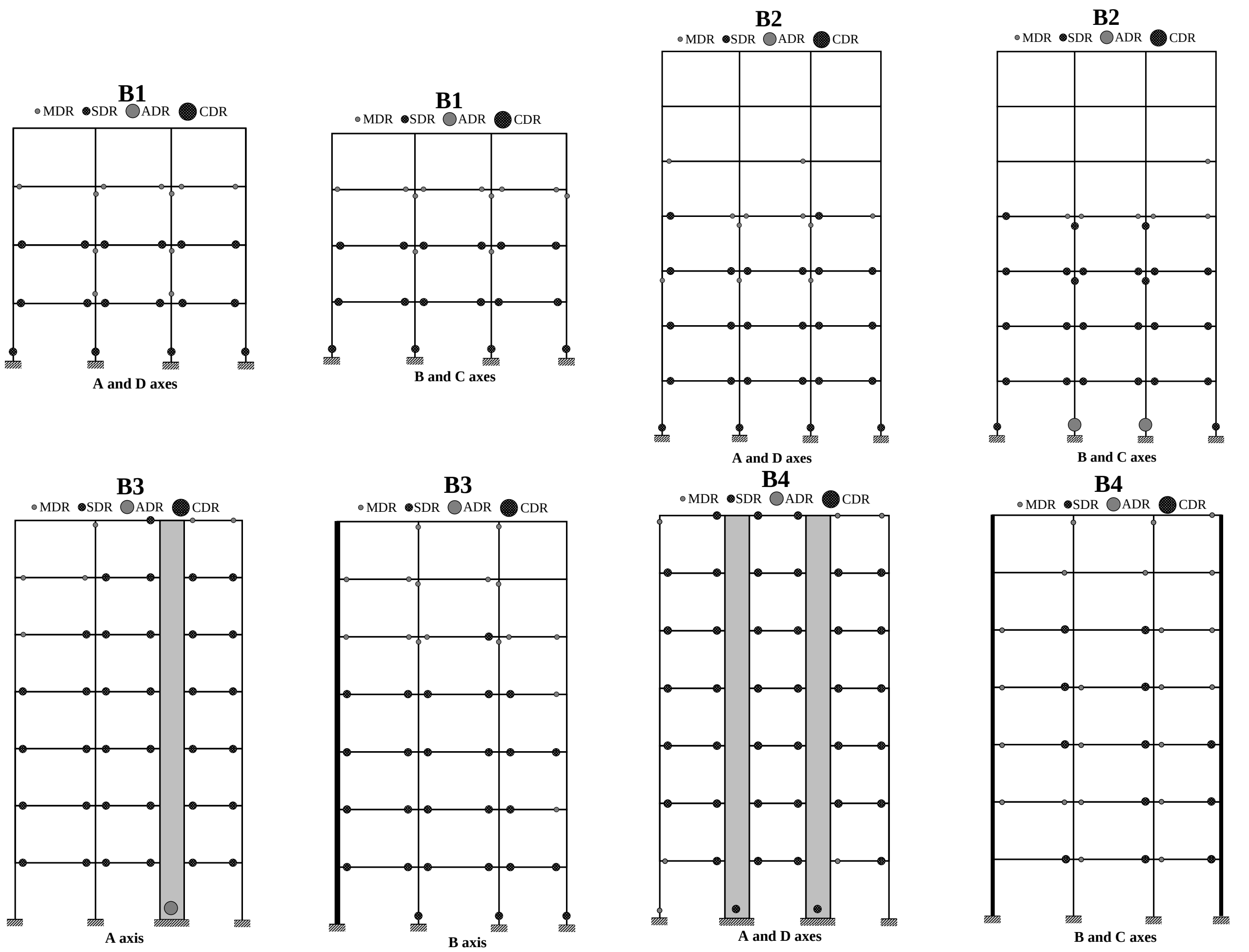


Fig. 15 Member damage ranges for E1 earthquake

observe plastic hinge distribution over the buildings (Fig. 15).

Since the capacity design philosophy was considered in the design of buildings, the plastic hinges were mostly formed at the ends of the beams, as expected (Fig. 15). Although the plastic hinges were formed at both ends of some columns, the minimum damage range (MDR) was not exceeded for these members (Fig. 15). It should be also noted that no sway mechanism in the buildings occurred due to formation of the plastic hinges.

As a major difference from other performance evaluation procedures, the building performance procedure in TEC uses the story based damage state (TEC 2007). The damage state of a story is evaluated by percentage of beam and column damage ranges (MDR, SDR, ADR, CDR). For this purpose, the ratio of beams in each damage range to total beam number in story is calculated. In columns, the ratio of total shear carried by the columns in each damage range to total story shear is calculated. The ratios calculated for each story are compared with the criteria given in Table 6 for each story (TEC 2007). The performance level corresponding to the most critical story is considered as the building performance level. The structural members of building (beam/column/shear-wall) were separately evaluated by means of the mentioned evaluation procedure. The obtained story performance levels and building performances for both earthquakes are given in Figs. 16 and 17, respectively.

Table 6 Story/building performance criteria in TEC (2007)

Performance level	Member type	
	Beams	Columns / Shear-walls
Immediate Occupancy (IO)	Members in the SDR ≤%10 Other members in the MDR	All members in the MDR
Life Safety (LS)	Members in the ADR ≤%30 Other members in the MDR or SDR	Members in the ADR≤%20 Other members in the MDR or SDR
Pre-collapse (PC)	Members in the CDR ≤%20 Other members in the MDR, SDR or ADR	All members in the MDR , SDR or ADR

"Life Safety" performance level was obtained for E1 earthquake in the building B1 (Fig. 17). The damage levels of 1st story beams and columns and 2nd story beams governed the building performance (Fig. 16). The 4-story high ductility moment resisting frame (B1) has achieved the target performance (Life Safety) predicted in TEC.

"Pre-Collapse" performance level was obtained for E1 earthquake in the building B2 (Fig. 17). The damages in the

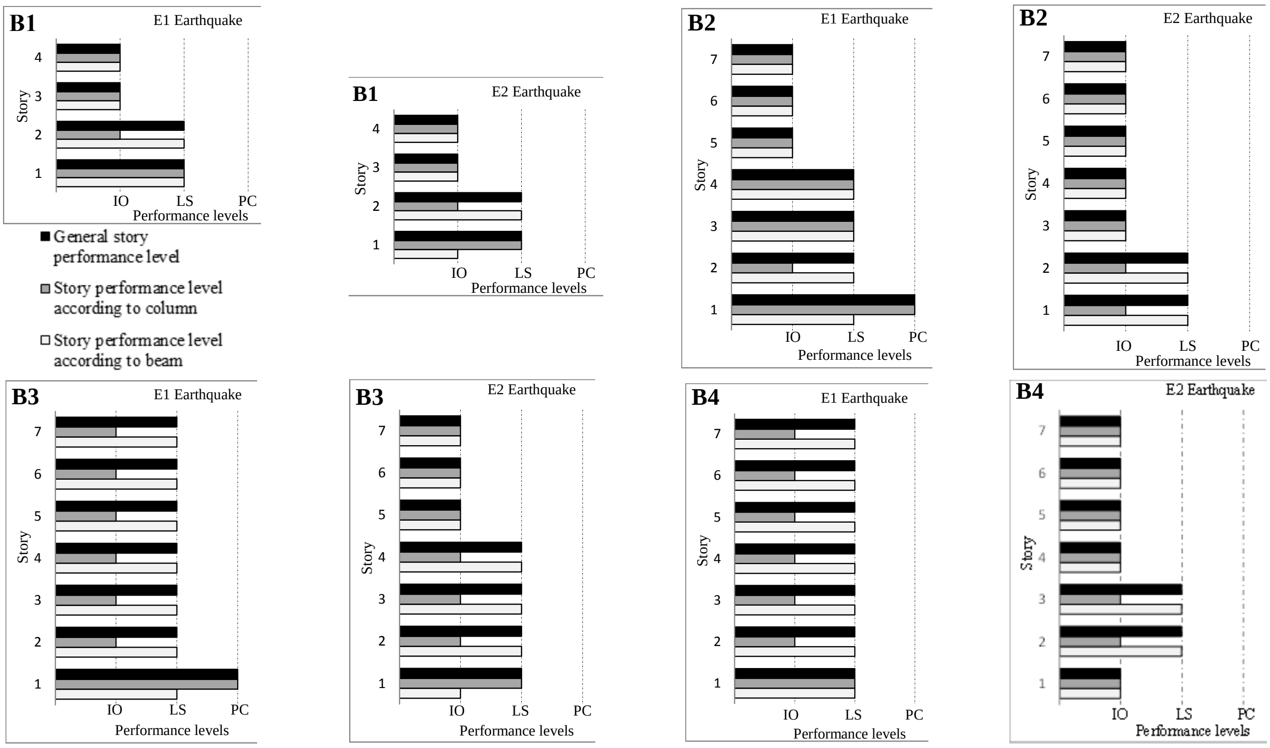


Fig. 16 Story / beam /column performance levels for E1 and E2 earthquakes

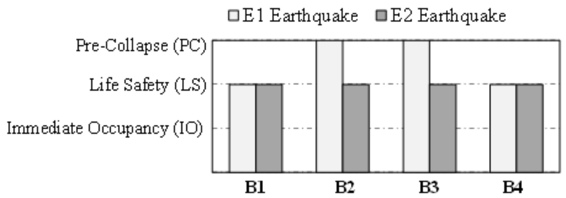


Fig. 17 Building performance levels for E1 and E2 earthquakes

Table 7 Ratios (%) of the damage ranges for the earth. E2

Story	Building B1				Building B2			
	Beam		Column		Beam		Column	
	MDR	SDR	MDR	SDR	MDR	SDR	MDR	SDR
1	100	0	68	32	0	100	100	0
2	67	33	100	0	0	100	100	0
3	100	0	100	0	100	0	100	0
4	100	0	100	0	100	0	100	0
5	-	-	-	-	100	0	100	0
6	-	-	-	-	100	0	100	0
7	-	-	-	-	100	0	100	0
	Building B3				Building B4			
Story	Beam		Column		Beam		Column	
	MDR	SDR	MDR	SDR	MDR	SDR	MDR	SDR
1	100	0	100	0	100	0	100	0
2	75	25	100	0	83	17	100	0
3	67	33	100	0	83	17	100	0
4	83	17	100	0	100	0	100	0
5	100	0	100	0	100	0	100	0
6	100	0	100	0	100	0	100	0
7	100	0	100	0	100	0	100	0

first story middle columns were decisive in the building performance (Fig. 16). The 7-story moment resisting frame (B2) has not achieved the target performance (Life Safety) for the design earthquake.

The “Pre-Collapse” performance level was obtained for E1 earthquake (Fig. 17) in the building B3. The damages in the first story shear-walls governed the building performance (Fig. 16). The 7-story dual frame (B3) with high ductility has not achieved the target performance (Life safety) for the design earthquake. In the building B4, “Life Safety” performance level was obtained for the earthquake E1 (Fig. 17). The beam damages and the first story shear-wall damages were effective in the building performance (Fig. 16). When comparing to the B3, the 7-story dual frame (B4) with a double shear wall area has achieved the “Life Safety” performance level.

For the E2 earthquake, which represents the moderate earthquake level, none of the investigated buildings was completely achieved the “Immediate Occupancy” level (Fig. 17). The member damages in the buildings were obtained within the Minimum Damage Range or the Significant Damage Range (Table 7). Significant damages were observed both in the beams and in the columns of the building B1. The addition of shear-wall to the system reduced the number of significantly damaged members (Table 7). The performance of the building B4 was very close to the “Immediate Occupancy” level compared to other buildings.

In buildings with wide-beam, the story drifts (ratio of relative column end displacements to story height) and stability of structural system are also important indicators

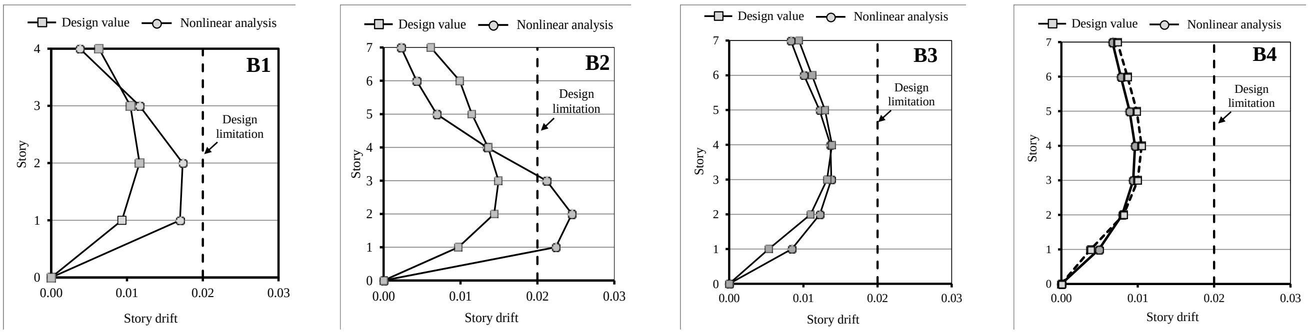


Fig. 18 Story drifts of the buildings

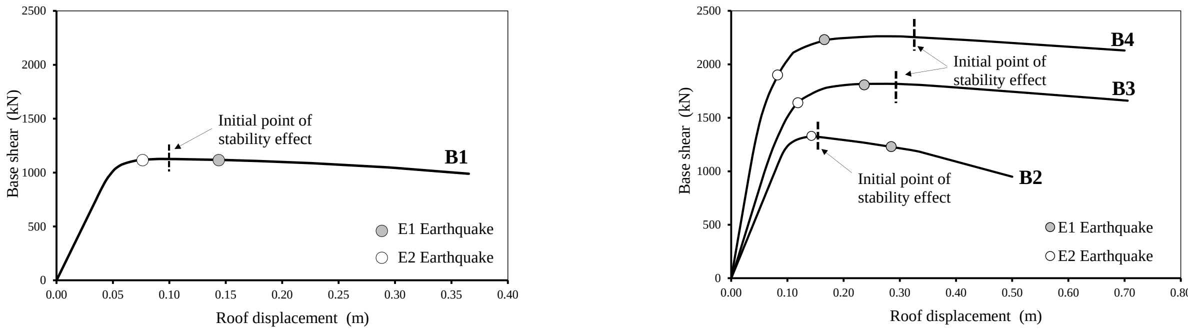


Fig. 19 Second-order effects on lateral load capacity of the buildings

for seismic response due to low lateral stiffness of these buildings. In this respect, the story drifts and second-order (p-delta) effects on the building capacity curve were assessed. The story drifts obtained by nonlinear analyses and the ones predicted in elastic design of the buildings were given comparatively in Fig. 18.

The nonlinear analysis results for the moment frame buildings (B1 and B2), showed that the displacement demands at the 1st, 2nd and 3rd stories were obtained well above the design values (Fig. 18). For the 7-story building (B2), the story drift limit of 2% in the TEC (2007) was exceeded at the lower stories, while the nonlinear analysis yielded lower drifts than the design values at the upper stories.

In the building with shear-wall (B3), the nonlinear analysis results showed that the displacement demands exceeded the design values at the lower stories. Very close drifts were obtained in other stories of this building (Fig. 18). The nonlinear analysis results and the design drift values were quite similar for the building B4 with shear-wall.

The points at which the degradation occurred due to the second-order effects were marked on the capacity curves in Fig. 19. Although the stability effects were observed before the design earthquake demand was reached in the 4-story building (B1), a significant degrading in the load capacity has not occurred.

In the 7-story building with the moment resisting frame (B2), the second-order effects were observed starting from the moderate earthquake demand. When the design earthquake demand was reached, a great degrading in the load capacity occurred in the building B2 (Fig. 19). In the buildings having dual frames (B3 and B4), the initial points of the stability effects moved forward and the capacity decrement completely disappeared at the design earthquake demand (Fig. 19).

5. Conclusions

In the study, seismic performance of low and medium-rise RC buildings with wide-beam and ribbed-slab were evaluated numerically. The moment resisting frame and dual frame (moment frame and shear-wall) structural systems were selected ascm structural systems of the buildings. Sufficiency of moment resisting frames designed with high ductility and necessity of shear-wall member were discussed for the buildings with wide-beam. The buildings were designed in accordance with the Turkish Earthquake Code (TEC 2007) and the performance evaluations were performed according to the strain-based nonlinear static method specified in the TEC. The second order (p-delta) effects on the lateral load capacity of buildings were also assessed in the study. The following conclusions can be drawn from the study.

- In the 4-story building (B1) representing low-rise building, the predicted earthquake performance in the TEC was achieved by using the moment resisting frame with high ductility. There was no significant stability problem (degradation in lateral load capacity) in the structural system for the considered earthquakes.
- In the 7-story building (B1) representing medium-rise building, the use of moment resisting frame with high ductility was inadequate both in terms of seismic performance and structural stability. The stability problem ceased to exist with the addition of shear-wall (about 0.6% of the building floor area) to the 7-story building. However, the predicted performance for the

severe earthquake was not achieved for this building. The desired performance level was provided when the shear-walls were added up to 1.2% of the building floor area.

• In the buildings with moment resisting frames (B1 and B2), the story drifts based on nonlinear analysis were considerably different from the design drifts and the difference increased in the 7-story building. In the buildings with dual frames (B3 and B4), the nonlinear analysis results were compatible with the design drifts.

• For the moderate earthquake E2, none of the investigated buildings was completely achieved the target performance level (Immediate Occupancy). In the buildings with dual frames, however, the member damage levels decreased comparing to the buildings with moment resisting frames for this earthquake level.

Consequently, the results of this study showed that use of moment resisting frames with high ductility is sufficient for the selected 4-story building with wide-beam, as is predicted in the TEC. On the contrary, the moment resisting frames with high ductility is not sufficient for the 7-story building and the shear-wall members should be added to structural system in sufficient quantity. Based on this, it can be concluded that the high ductility requirement of the TEC could not be adequate alone for medium-rise buildings with wide-beam in the high earthquake regions. The TEC permits the limited ductility frame up to 13 m height in the moderate earthquake regions. It is required to use dual frames for taller buildings. Based on the finding of study, a similar approach is recommended for the wide-beams buildings in the high earthquake regions.

In this respect, the necessary building height limits and required shear-wall ratios should be investigated extensively for medium-rise buildings with wide-beam and ribbed-slab. In addition, it would be useful to investigate the irregular buildings, infill-wall effects and hysteretic effects.

References

ACI 318 (2008), Building Code Requirements for Structural Concrete and Commentary, American Concrete Institute, Farmington Hills, MI, USA.

ASCE/SEI41-13 (2013), Seismic Evaluation and Retrofit of Existing Buildings, American Society of Civil Engineers Virginia, USA.

Benavent-Climent, A. (2005), "Shaking table tests of reinforced concrete wide beam-column connections", *Earthq. Eng. Struct. Dyn.*, **34**(15), 1833-1839.

Benavent-Climent, A., Cahis, X. and Vico, J.M. (2010), "Interior wide beam-column connections in existing RC frames subjected to lateral earthquake loading", *Bull. Earthq. Eng.*, **8**(2), 8(2), 401-420.

Cosar, A., Kocak, A., Guney, D., Selcuk, M.E. and Yıldırım, M. (2011), "23 October 2011 Van earthquake technical investigation report", Yildiz Technical University, Istanbul, Turkey.

Donmez, C. (2013), "Seismic performance of wide-beam infill joist block RC frames in Turkey", *J. Perform. Constr. Facil.*, **29**(1), 04014026.

Elsouri, A.M. and Harajli, M.H. (2013), "Behavior of reinforced concrete wide concealed-beam/narrow-column joints under lateral earthquake loading", *ACI Struct. J.*, **110**(2), 205-215.

Eurocode-8 (2008), Design of Structures for Earthquake Resistance-Part 1: General Rules, Seismic Actions and Rules for Buildings, European Committee for Standardization, Brussels.

Gentry, R.G. and Wight, J.K. (1994), "Wide beam-column connections under earthquake-type loading", *Earthq Spectra*, **10**(4), 675-703.

Gentry, T.R. (1992), "Reinforced concrete wide beam-column connections under earthquake-type loading", Ph.D. Thesis, Univ. of Michigan, Ann Arbor, Michigan, U.S.A.

Goldsworthy, H.M. and Abdouka, K. (2011), "Displacement-based assessment of non ductile exterior wide band beam-column connections", *J. Earthq. Eng.*, **16**(1), 61-82.

Gulkan, P. (1998), "Turkey earthquakes June 27 1998 Ceyhan Misis earthquake reconnaissance report, METU-DMC", METU-EERC, Ankara, Turkey.

Gungor, I. (2014), "Numerical investigations on seismic performance of RC buildings with ribbed slab and wide beam", M.Sc. Dissertation, Institute of Science, Balıkesir University, Turkey.

Kulkarni, S.A. and Li, B. (2008), "Seismic behavior of reinforced concrete Interior wide-beam column joints", *J. Earthq. Eng.*, **13**(1), 80-99.

Lopez-Almansa, F.L., Domínguez, D. and Benavent-Climent, A. (2013), "Vulnerability analysis of RC buildings with wide-beams located in moderate seismicity regions", *Eng. Struct*., **46**, 687-702.

Malley, J.O., Celebi, M., Bruneau, M., Saaticioglu, M., Erdik, M. and Gulkan, P. (1993), "1992 Erzincan earthquake: Buildings", *Earthq. Spectra*, **9**(S1), 49-85.

NZSA (1995), Standard for the Design of Concrete Structures (NZS3101-95), New Zealand Standards Authority, New Zealand.

Paulay, T. and Priestley, M.J.N. (1992), *Seismic Design of Reinforced Concrete and Masonry Buildings*, John Wiley and Sons Inc., New York, USA.

Quintero-Febres, C.G. and Wight, J.K. (2001), "Experimental study of reinforced concrete interior wide beam-column connections subjected to lateral loading", *ACI Struct. J.*, **98**(4), 572-582.

SAP2000-V8.1.2 (2008), Structural Analysis Program, Computers and Structures Inc., Berkeley, California, USA.

Stehle, J.S., Goldsworthy, H. and Mendis, P. (2001), "Reinforced concrete interior wide-band beam-column connections subjected to lateral earthquake loading", *ACI Struct. J.*, **98**(3), 270-279.

TEC (2007), Turkish Earthquake Code, The Minister of Public Works and Settlement, Ankara, Turkey.

TS 500 (2000), Turkish Standard: Requirements for Design and Construction of Reinforced Concrete Structures, Institute of Turkish Standards, Ankara, Turkey.

XTRACT (2004), Cross Section Analysis Program of Structural Engineers, Imbsen Software Systems.

CC