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The Effect of Column Cross Section to Frame Ductility in RC Frames Having Poor Seismic Performance

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Abstract: This study deals with an experimental study on the effect of column cross section to frame ductility in RC frames. 2-bay, 2 story, 1/3 scale reinforced concrete frame specimens which were designed and constructed with deficiencies as observed in practice of Turkey were tested under reversed-cyclic lateral loading and low level axial loading. The difference between two specimens is column cross section area. The behaviour characteristics obtained from specimens at the end of the tests are determined; in addition, considering parameters like system ductility, strength and stiffness. In the analytical part of this study, pushover analyses were performed for two frame specimens. The results of tests and pushover analysis are compared between each other and evaluated. It is concluded that frame lateral ductility is related with column cross section.

Key words: Reinforced concrete frame, ductility, non-seismic detailing, reversed-cyclic lateral loading, pushover analysis

INTRODUCTION

The poor performance of the old RC structures with substandard reinforcing details had been widely observed during recent earthquakes in Turkey. The characteristics of typical buildings in Turkey include flexible columns, soft stories and non-seismic detailing (Adalier and Aydingun 2001, Aydan 1997, Dogangun, 2004). Leave the non-engineered buildings aside, engineered structures in Turkey are far from possessing qualities that would ensure satisfactory seismic performance. Although Turkey has a developed seismic code called "The 2006 Turkish Earthquake Code" (TEC-2006), which was prepared to ensure that all structures have adequate stiffness, strength and ductility, but most of the structures do not have these properties.

Ductility is one of the most important parameter for RC structures to resist earthquake effects. Especially, adequate transverse reinforcement, compression steel bars, low-level axial load and adequate column cross section area is help to form plastic hinge and therefore, the system behaves more ductile. Most of the reinforced concrete buildings in Turkey do not have adequate ductility. In the last decade, after the major earthquakes, the main observation is that (EERI Special Earthquake Report, 1999; Bruneau, 2002; Sezen *et al*., 2003; Cagatay, 2005) the majority of moment-frame component damages occurred in columns. These column damages were due to unsuitable transverse reinforcement, inadequate cross section dimensions, non-ductile details, unconfined lap splices, inadequate anchorage lengths and excessive beam strength (Celep and Ozer, 1998; Igarashi, 1999).

In the past, different researchers have conducted various researches on the performance and ductility of reinforced concrete columns and frames. However, attention concentrated mainly on detail practice in codes, particularly American Concrete Institute (ACI) 318-05 code, UBC or EC8 (Eurocode 8). But the practice in reinforcement detailing, construction and the application in Turkey are different from the seismic code regulations, because of uncontrolling.

The main aim of this study, is to investigate the effect of column cross section and longitudinal steel reinforcement ratio to frame ductility in reinforced concrete frames having poor seismic performance. The main difference between specimens is column cross section area. For this purpose, there were carried out experimental studies in the Earthquake Research Laboratory of Civil Engineering Department of Selcuk University. Two number of 2 bay-2 story 1/3 scaled reinforced concrete frame specimens were designed and constructed with deficiencies as observed in practice. The specimens were tested under reversed-cyclic lateral loading. Loading pattern was applied load controlled up

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to yielding and then displacement controlled up to collapse. Theoretical part of this study was performed by using pushover analysis. Test results were evaluated according to system ductility, strength and stiffness.

IMPORTANCE OF DUCTILITY

Ductility is considered as the ability of a structure to undergo large plastic deformations without losing strength. There exist studies about material, cross section, member and system ductility in literature (Kappos *et al.* 1999; Gioncu, 2000). Here for system ductility,

$$\mu_\delta = \frac{\delta_u}{\delta_y} \tag{1}$$

"general" relations can be written (Eq. 1). As understood from the formulation, system ductility is expressed in terms of displacement ductility in reinforced concrete structures. The system ductility can be expressed as the ratio of maximum lateral displacement under limit condition to the lateral displacement at the time of yielding. There are various assumptions for yield displacement and the failure criteria in the literature. Many researchers accept that the yield displacement is equal to the displacement corresponding to 75% of maximum lateral force (P_{75}, δ_{75}) and the failure force of system is 85% of maximum lateral force (P_{85}, δ_{85}) (Lu *et al.*, 2001) (Fig. 1).

The frame (system) ductility that will be examined in this study, is an extremely significant concept to determine the structure behaviour coefficient (load reduction factor, response modification coefficient) R (or q) being one of the most important parameters for structural calculation ($\mu_\delta \approx R$). Modern seismic codes of practice have adopted the concept that certain structural damage can be tolerated during major earthquakes, provided that structures are adequately ductile. In a ductile structure, the inelastic energy dissipation can be achieved in a somewhat regulated manner without jeopardizing the integrity or stability of the structural system. The design seismic force can be drastically reduced and this renders the ductile design generally economical. In seismic regulations in Turkey (TEC-2006) and other countries (UBC-97, Eurocode-8-98), the R coefficient depends on the assumption of decreasing elastic seismic loads that will act on the structure by assuming structure's ductile behaviour. Especially in recent years, the studies about the calculation of frame ductility (μ_δ) are carrying great importance for determining structure behaviour coefficient R that is significant for

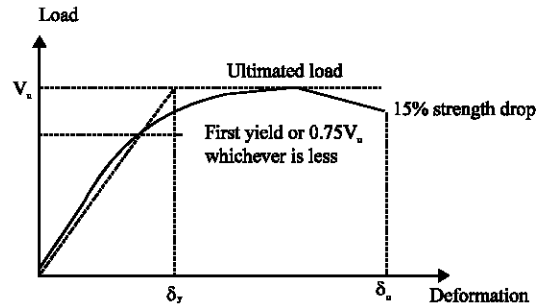


Fig. 1: Force-displacement relationship of a frame

earthquake engineering and mentioned in seismic regulations of the entire world (Borzi and Elnashai, 2000).

EXPERIMENTAL PROGRAMME

Properties of test specimens: The study represents material and section properties of test specimens, test set-up and location of transducers are presented. In this study, 1/3 scaled two stories two bay reinforced concrete frames having non-seismic detailing were tested under reversed-cyclic lateral loading. These frames have deficiencies commonly observed in Turkey, such as low concrete compression strength, inadequate lateral stiffness, inadequate confinement, lapped splices floor levels and using plain bars. The dimensions of the test specimens with displacement transducer locations are given in Fig. 2 and the reinforcement layouts of RC frames are given in Fig. 3.

B1 frame B2 frame: The main purpose of these tests is determining the effect of column cross section on frame ductility. The cross sections of these columns and beams are shown in Fig. 4 and 5, respectively. In all columns, 8 mm diameter plain bars were used as longitudinal reinforcement. In all beams 6 mm diameter plain and bent-up bars were used as beam reinforcement. Bended beam bars have been used commonly in Turkey. Changing region of bended beam bars were placed at 1/7 portion of design span far from column face at side spans and 1/5 portion at middle spans. In all beams and columns, 4 mm diameter stirrups spaced at 100 mm were used. The ends of these ties had 90 degree hooks to represent common practice in Turkey although requirement of using 135° hook at stirrups emphasizes in TEC2006. The column longitudinal reinforcements were spliced at floor and foundation levels.

$$M_{rc} / M_{rb} > 1.2 \tag{2}$$

In this formula, M_{rc} and M_{rb} represent ultimate strength of column and beam, respectively.

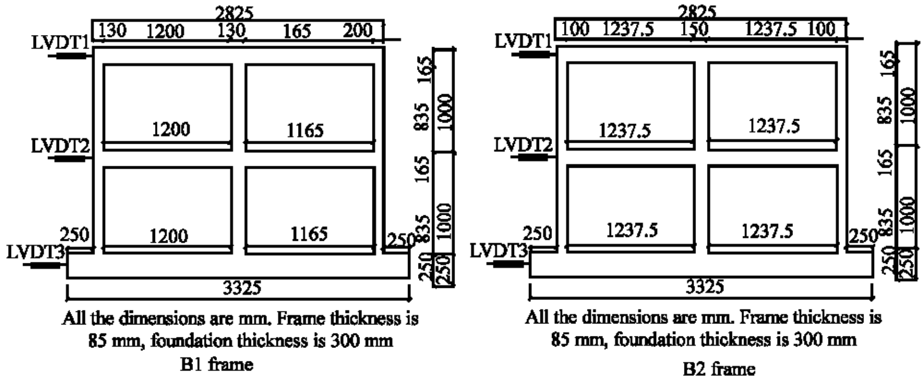


Fig. 2: The dimensions of the test specimens and transducer locations

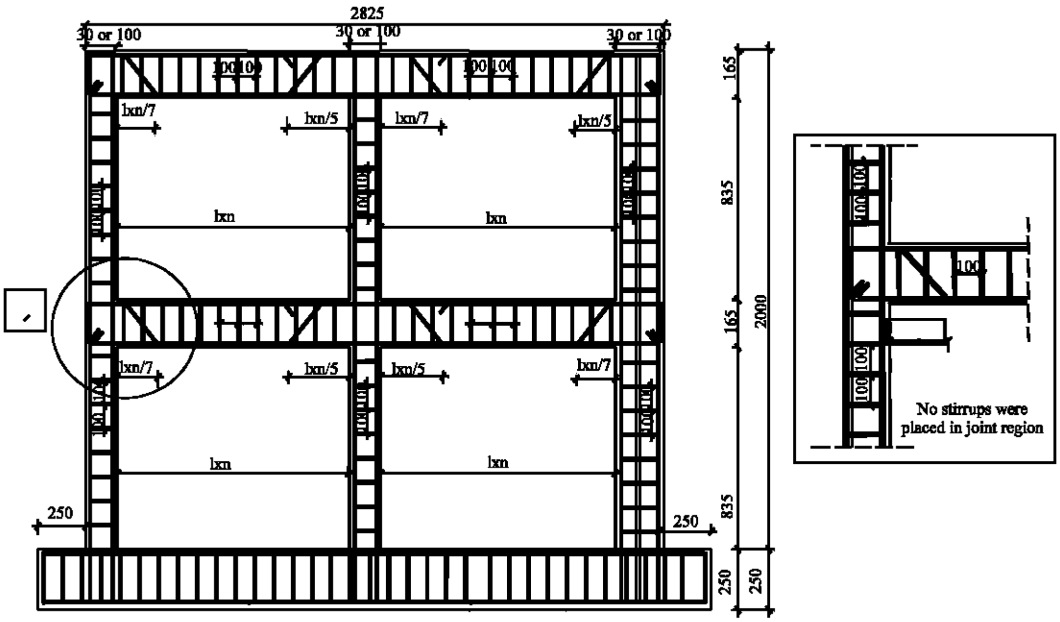


Fig. 3: Reinforcement layouts of RC frames(All the dimensions are mm) *All the dimensions are mm

Table 1: Mixture proportion for test frames (weight for 1 m^3 of concrete)

Material	Weight, (kg)	Proportions by weight, %
Cement	240	11.00
0-7 mm aggregate	1810	80.00
Water	216	9.00
Total	2266	100.00

Table 2: Concrete compression strength of frame specimens

Frame type	$f_{c,28th}$ (MPa)	Average of $f_{c,28th}$ (Mpa)
B1	16.55	14.24
	12.40	
	13.77	
B2	12.20	13.20
	12.90	
	14.50	

In Fig. 3 detail A, the joint region is shown.

Concrete compression strength changes between 10-15 MPa in Turkey. In this study, it was determined by average of three cylindrical concrete specimens for one frame according to TBC-500-2000. In Table 1, the concrete mixture proportions of tested frames are presented. Materials used in this mixture are given by weight for 1 m^3 concrete. Target cylinder concrete compression strength was selected 12 MPa for represent to the Turkish building construction before TEC-75. The obtained concrete compression strengths of frames is given in Table 2. Yielding and ultimate strength values of reinforcements used in these specimens are listed in Table 3.

Table 3: Yielding and ultimate strength values of reinforcements

Bar diameter, mm	f_y, (MPa)	f_{su}, (MPa)	Type
4	333	469	Plain
6	541	638	Plain
8	447	653	Plain

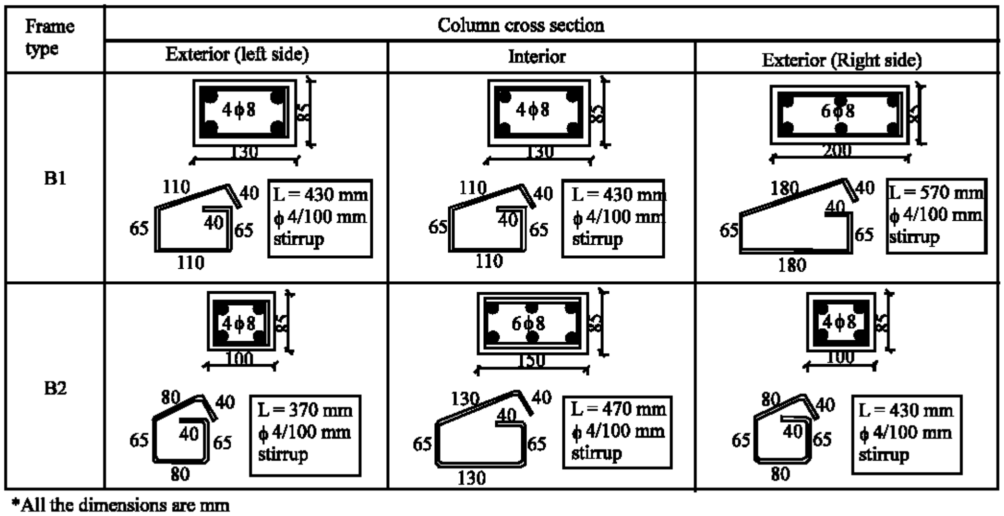


Fig. 4: Column cross sections

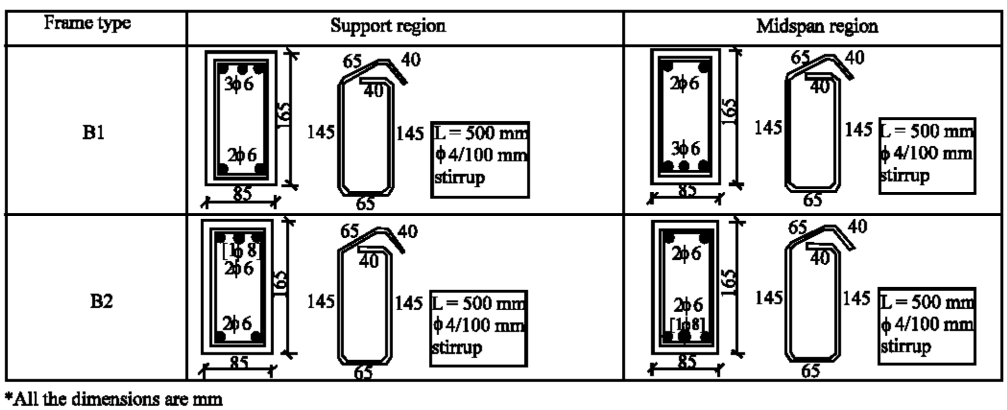


Fig. 5: Beam cross sections

Test devices: Concrete of test specimens was in situ in horizontal position in Earthquake Research Laboratory of Selcuk University and kept in air temperature for 28 day as all cylindrical concrete specimens. Test specimens were held up vertical position by special steel cages and were tested at the same position. The testing system consisted of the strong floor, reaction wall, loading equipment, instrumentation and data acquisition system (Fig. 6). The foundation of the test specimens were fixed to the strong floor by high-strength steel bolts. The specimens were tested under reversed-cyclic lateral loading. Lateral load was applied at the top-story floor level by using a hydraulic jack to measure the magnitude of this load and this load was transmitted to the frames by means of special transmission bolts. A steel stability frame was constructed around the test specimens to prevent out-of-plane displacements. Lateral load was measured by compression-tension loadcell which had 500 kN capacity.

Axial load was measured by one directional loadcell which had 200 kN capacity and was controlled continuously. The axial load was applied on each column approximately $0.10A_cf_c$ was applied on columns by steel cables placed in a bobbin system prior to application of the lateral load. These limit values were determined according to the biggest column cross section for each test specimen. The lateral displacements of the test specimens at each floor Level were measured by Displacement Transducers (LVDT). The lateral loading programme was applied load controlled up to yielding and after this point it was applied displacement controlled.

BEHAVIOUR OF SPECIMENS AND TEST RESULTS

General behaviour: Observed failure mode occurred by flexure effect in both frames. The maximum lateral load was measured 47.61 kN in B1 frame and 50.07 kN in B2 frame.

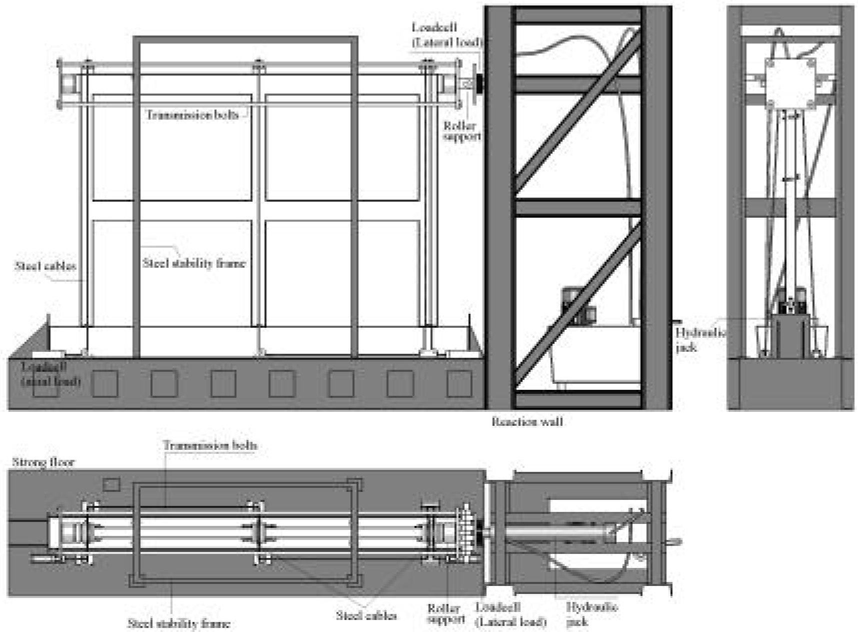
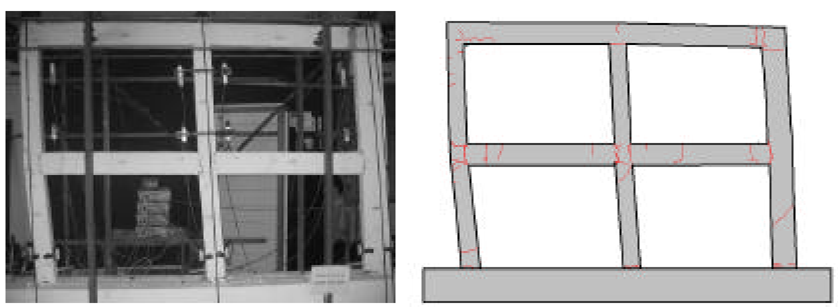
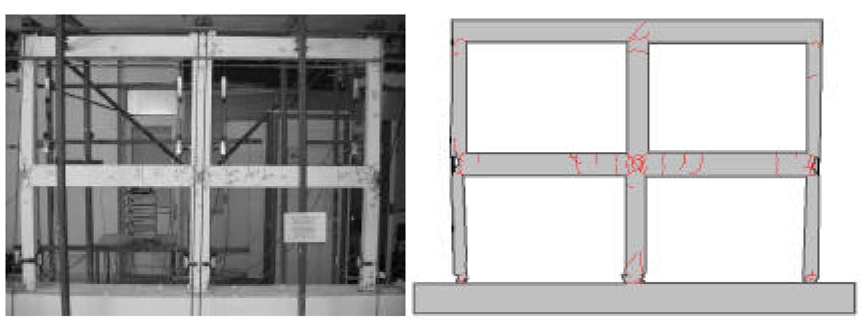


Fig. 6: Test set-up and loading system



B1 frame



B2 frame

Fig. 7: Crack pattern of tested frames

At 15 kN lateral load level, first crack occurred in right support of right first story beam. The first crack was observed in 20.72 kN load level (forward cycle) at the middle first story column base of B1 specimen. On the other hand, the first crack occurred at left support of left first story beam at 15 kN. The first crack was observed in 25 kN load level (forward cycle) at right column base of B2 specimen. The maximum value of story drift ratio is 0.0035

Table 4: Measured values corresponding to yielding and ultimate level

Loading direction	Specimen	Yielding load, P_y(kN)	Maximum load, P_{max} (kN)	Yielding displacement δ_y (mm)	Maximum load displacement δ_{max}(mm)	Ultimate displacement δ_{ult} (mm)	δ_{85} or δ_{ult}(mm)	$P_{\delta,0.0035}$ (kN)
Forward	B1	31.45	41.93	13.01	38.12	102.03	91.01	33.30
	B2	33.71	44.95	15.50	45.48	83.11	63.92	18.50
Backward	B1	35.71	47.61	8.51	51.60	94.17	94.17	22.45
	B2	37.55	50.07	17.73	50.24	93.08	80.73	20.23

Table 5: Test and pushover analysis results

Loading direction	Specimen	P_y(kN)		P_{max} (kN)		Yielding load, δ_y(mm)		Maximum load, δ_{85} (mm)		$\mu = \frac{\delta_{85}}{\delta_y}$	
		Test	Theor.	Test	Theor.	Test	Theor.	Test	Theor.	Test	Theor.
Forward	B1	31.45	36.33	41.93	44.51	13.01	10.59	91.01	43.59	6.99	4.11
	B2	33.71	39.59	44.95	49.56	15.50	14.13	63.92	46.98	4.12	3.32
Backward	B1	35.71	33.97	47.61	44.75	8.51	9.42	94.17	44.85	11.06	4.76
	B2	37.55	39.59	50.07	49.56	17.73	14.13	80.73	46.98	4.55	3.32

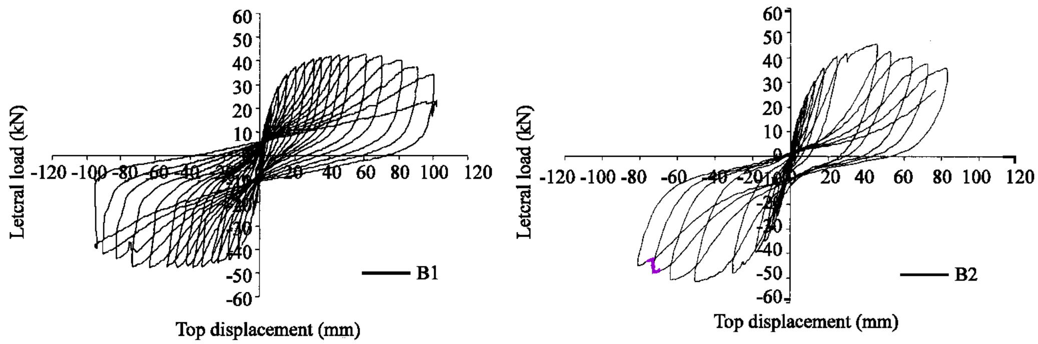


Fig. 8: Hysteretic curves of test specimens

according to TEC-98. The load at this limit displacement level in B1 and B2 frame is 33.30 kN and 18.50 kN at forward cycle and 22.45 and 20.23 kN at backward cycle, respectively. First vertical crack on beams occurred in interrupting zone of bent-up bar at 37.49 kN load level (forward cycle) on B1 frame and at 30 kN load level (backward cycle) on B2 frame. In following cycles, horizontal and diagonal cracks occurred in the first story left column-to-beam connection in both frames. Cracks increased rapidly after cracking of column bases had been developed. In this phase, loading type was applied as displacement-controlled. In Fig. 7, crack pattern of tested frames at the end of the tests were shown.

Shear failures were observed on exterior column-beam joints due to the lack of confinement and flexural failure occurred in changing region of bended beam bar. Column longitudinal reinforcement buckled and concrete cover crushed due to reinforcement hook at side joints of B2 frame. In foundations, no cracks or concrete crushing was observed in both frames as expected.

There was no out-of-plane action in both frames, because special arrangements were used to prevent this effect. No compression failure was observed in any column because of low axial load level. Vertical cracks occurred in changing region of bended beam bars of first story beams because of reversed-cyclic moments. This situation shows that beam tension reinforcement shouldn't cut at support regions.

Table 4 shows yielding load, ultimate load, displacement values and displacement ductility ratio. In B1 frame backward cycle, lateral displacement hadn't reached δ_{85} value, so δ_{ult} had been considered in ductility ratio computing for this specimen (Table 5). δ_{85} displacement value is corresponding to 85% of maximum lateral load and was assumed to failure displacement of frames.

Figure 8 shows the hysteretic curves obtained from frame tests under reversed-cyclic lateral loading. Response envelope curves for specimens were plotted by connecting the peak points of these hysteretic curves for each specimen. Response envelope curves showed the strength and stiffness characteristics of the specimens and their general behaviour (Fig. 9). Although the 2 response envelope curves have a same shape, the initial stiffness values of specimens had shown difference. Initial stiffness was calculated as the slope of load displacement curve in the first cycle. The stiffness values were determined 17.11 kN mm^{-1} and 5.90 kN mm^{-1} at B1 and B2 frames, respectively (Fig. 10).

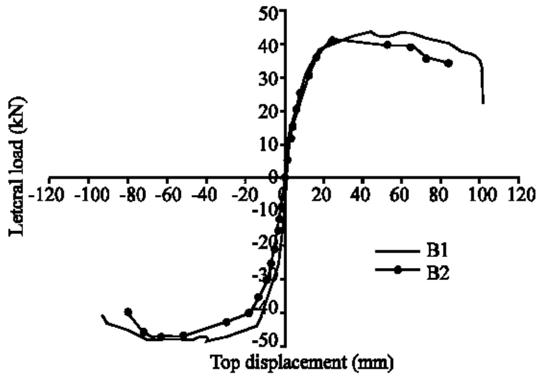


Fig. 9: Response envelope curves of test specimens

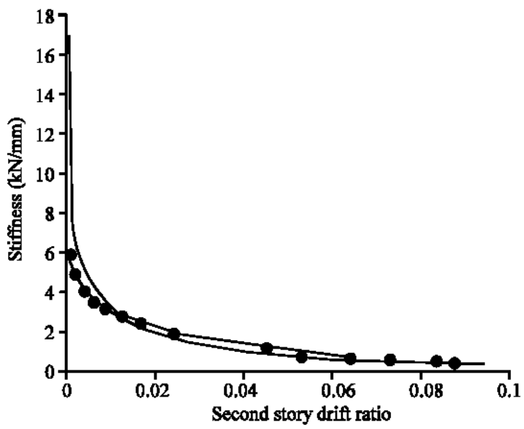
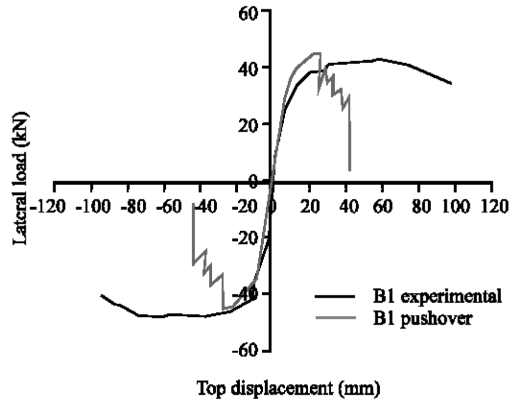


Fig. 10: Stiffness degradation curves of test specimens

THEORETICAL STUDY

Modal analysis (pushover analysis): In the analytical stage, the main purpose is to obtain the capacity curve (load-displacement curve) of each frame. In order to determine the capacity curves of these structures beyond the elastic limits, the pushover analysis is required. Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. In recent studies that have been based on the structural behaviour, the pushover analysis method has become the powerful method for the seismic design (Makarios, 2005; Inel and Ozmen, 2006).

After the computer model of the building has been prepared to perform non-linear pushover analysis, hinge properties of the components should be determined. Hinge properties contain the plastic rotation values that a component's end can carry and acceptable plastic rotation values for the performance level. In ATC-40 and FEMA-356, hinge properties are given according to the



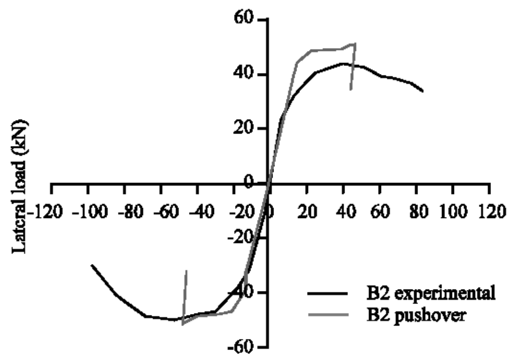


Fig. 11: Response envelope curves for two specimens

component type and failure mechanism. After determining of the horizontal and vertical load's values, internal forces which necessary to determine the hinge parameters of the components of tested frames, are determined according to FEMA-356. After all hinges are placed at their components' ends, the pushover cases are defined in the software. The software used for these processes was SAP2000. The program uses a series of sequential elastic analysis, superimposed to approximate a force displacement capacity diagram of the overall structure. In the model, the control nodes are mass concentrated sections. The lateral load was applied to the second floor beam axe and to one of the mass concentrated section. Also, the lateral force in the selected pattern is applied to the structure in a stepwise manner. The total base shear starts from zero and increases. In each step, the internal member forces are calculated and the top displacement and the base shear force are recorded to plot the capacity curve. Loading type was displacement controlled. According to the lateral load-top displacement curve for each mentioned frame is obtained to examine frame behaviors. In this study, pushover analysis was performed for two test specimens. In Fig. 11, response envelope curves obtained from pushover analysis and tests are shown comparatively and results are presented in Table 5. It is clearly shown that B1 frame was more ductile than B2 frame.

CONCLUSION

This study is carried out to investigate the effectiveness of the column cross section to frame displacement ductility which is the most important parameter of seismic design. This study consists of the experimental and analytical parts. In the experimental program, test specimens were designed to represent the RC buildings stocks in Turkey. Based on the experimental and analytical results, the following conclusions were drawn:

- The damage type of test specimens was same as the actual damage type of RC structures after earthquakes.
- Changing of column cross section area is basic design parameter of this study and this caused significant effect on frames stiffness. Initial stiffness of B1 was 2.90 times greater than B2's.
- Since the amount of column longitudinal reinforcement bars in two specimens is same, the lateral load capacity of the frames is very closely (about 7% differences). It shows that the most important parameter for calculation of frame's ultimate capacity is column longitudinal reinforcement.
- The first cracks developed at the end of the first story beams in two specimens. The other developing cracks continued on the beams and load-displacement curve slope did not decrease until first crack was observed on columns.
- Vertical cracks on beams were shown at bended beam bar changing regions. The behaviour shows that the bended beam bars have no effectiveness under reversed-cyclic lateral loading which simulate earthquake load.
- Bended beam bars makes the frame damage more heavily than expected.
- The shear failure (X cracks) occurred on the boundary column-to-beam connection zones on the each specimen because of no adequate lateral ties (transverse reinforcement).
- It was interesting to observe that at the maximum displacement level of frames according to the TEC-2007 (7 mm), the damage level of B1 frame is less than B2 frame's. Because column's cross sections of B1 frames are much bigger than B2 frame's.
- The backward and forward loading displacement ductility of frames is related to the column cross section areas. The ductility ratio of frames (B1/B2) is about 1.69 for forward loading and 2.43 for backward loading based on experiments. On the other hand, the determined ductility ratio of frames (B1/B2) according to analytical pushover curves is about 1.24 for forward cycle and 1.44 for backward cycle.
- According to analytical part of this study, even though the variation in the base shear capacity between the test results and pushover analysis is about 10 %, the SAP2000 program successfully simulated the ultimate strength.

Ductility is one of the most important parameter for RC structures to resist earthquake effects. Most of the reinforced concrete buildings in Turkey and other developing countries do not have adequate ductility. After the major earthquakes, the main observation is that the majority of moment-frame component damages were occurred in columns. The tests reported in this paper demonstrated the importance of relation between column cross-section and displacement ductility. The increasing of column cross sections in multi story-multi bay RC frames having same properties causes great improvement on displacement ductility.

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