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Determination of lateral strength and ductility characteristics of existing mid-rise RC buildings in Turkey

Taner Ucar[*1], Onur Merter[2a] and Mustafa Duzgun[2b]

[1]*Department of Architecture, Dokuz Eylul University, Izmir, Turkey*
[2]*Department of Civil Engineering, Dokuz Eylul University, Izmir, Turkey*

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Abstract. This paper presents a comprehensive work on determination of yield base shear coefficient and displacement ductility factor of three to eight story actual reinforced concrete buildings, instead of using generic frames. The building data is provided by a walkdown survey in different locations of the pilot areas. Very detailed three dimensional models of the selected buildings are generated by using the data provided in architectural and reinforcement projects. Capacity curves of the buildings are obtained from nonlinear static pushover analyses and each capacity curve is approximated with a bilinear curve. Characteristic points of capacity curve, the yield base shear capacity, the yield displacement and the ultimate displacement capacity, are determined. The calculated values of the yield base shear coefficients and the displacement ductility factors for directions into consideration are compared by those expected values given in different versions of Turkish Seismic Design Code. Although having sufficient lateral strength capacities, the deformation capacities of these typical mid-rise reinforced concrete buildings are found to be considerably low.

Keywords: existing reinforced concrete buildings; nonlinear static analysis; characteristic points of capacity curves; yield base shear coefficient; displacement ductility factor

1. Introduction

Understanding the nonlinear response and damage characteristics of reinforced concrete (RC) buildings subjected to significant earthquakes is essential for assessment of seismic performance of existing buildings, as well as safe and economic design of new buildings. Nonlinear static pushover analysis, which considers the inelastic behavior of the structure, is a simple and a practical tool to compute seismic demands imposed by the design earthquake on the building and its structural components. In most cases, nonlinear static pushover analysis provides adequate information on the strength and deformation capacities of buildings in post elastic range.

Determination of the yield base shear capacity and two displacement parameters, named as the yield displacement and the ultimate displacement capacity of buildings are of essential importance

∗Corresponding author, Assistant Professor, E-mail: taner.ucar@deu.edu.tr
[a]Ph.D., E-mail: onur.merter@deu.edu.tr
[b]Professor, E-mail: mustafa.duzgun@deu.edu.tr

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in displacement-based performance assessment of buildings. The yield displacement represents significant yielding of the system when the yield base shear capacity of the building is attained and the ultimate displacement corresponds to the state at which the building reaches its deformation capacity (Akkar *et al*. 2005). The above mentioned displacement quantities are also generally used to represent the displacement-based structural damage limit states of seismic fragility curves, which provide a probabilistic evaluation of structural damage. The simplest damage variables are those based on the maximum inelastic deformation (Ellingwood 2001).

Several investigations are available dealing with the determination of the yield base shear capacity, the yield displacement and the ultimate displacement capacity of RC buildings. Definitions for the required and available ductility used in seismic design were discussed, methods for estimating the yield deformation and the maximum available deformation were described and a quasi-static procedure for establishing the available ductility factor of a subassemblage by laboratory testing was recommended (Park 1988, 1989). Uang (1991) derived the basic formulas for establishing the response modification factor R and the displacement amplification factor C_d used in the National Earthquake Hazards Reduction Program (NEHRP) recommended provisions. Seismic force reduction factor (FRF) and displacement amplification factor (DAF), which are functions of the structural ductility factor and structural overstrength factor, were derived and the appropriate relationship between FRF and DAF was verified from dynamic analyses of two instrumented buildings in California (Uang 1992). Uang and Maarouf (1994) evaluated the ratio between seismic deflection amplification factor and force reduction factor based on the dynamic responses of four actual building frames subjected to a set of eight historical earthquake records. Lam *et al*. (1998) presented new trends in the relationship between the ductility reduction factor and the ductility demand in the seismic design of buildings. The ductility-dependent and the overstrength-dependent behavior factors were estimated on the basis of corresponding inelastic spectra and using both static and dynamic inelastic procedures (Kappos 1999). Lee *et al*. (1999) determined the ductility factor considering five different hysteretic models. Inelastic constant ductility acceleration spectra were derived using two models: an elastic perfectly plastic representation and another more complex system which has a yield point, a maximum force point and a post-ultimate branch (Borzi and Elnashai 2000). A scaling model, which considers damage and ductility supply ratio, was proposed for estimating the damage-based strength reduction factors (Tiwari and Gupta 2000). Mwafy (2000) and Elnashai and Mwafy (2002) investigated the relationship between the lateral capacity, the design force reduction factor, the ductility level and the overstrength factor. The lateral capacity and the overstrength factor were estimated by means of inelastic static pushover and time-history collapse analysis for 12 buildings RC buildings with various characteristics. Paulay (2003) investigated seismic displacement capacity of RC building systems and researched the yield displacements, strength-displacement relationships, displacement ductility capacities and displacement profiles of RC walls, frames and frame-wall dual systems.

Additionally to previous studies, in last decade, deterministic and probabilistic, both analytic and experimental research on determination of the seismic capacity and the displacement ductility of RC buildings are also common trend. Tjhin *et al*. (2004) presented an analytical study to estimate the yield displecement of ductile RC wall buildings and “yield point spectra” were created to determine yield points of structural RC walls for a desired performance level. Faella *et al*. (2004) assessed the dependence of the performance point value on parameters controlling the bilinear relationship and on conversion procedure. Reliability and accuracy of procedures based on the use of constant ductility spectra was assessed by comparing results with the ones from nonlinear dynamic analyses. Sullivan *et al*. (2004) reviewed four of the most recent displacement-based design methods that utilize response spectra, two of which were initial stiffness based and two of which were secant stiffness based and the performance of each procedure was assessed by means of non-linear time history analyses. Krawinkler and Nassar (2005) studied seismic design of structures based on ductility and cumulative damage demand and presented a seismic damage procedure which explicitly considers the ductility concept. Varela *et al*. (2006) proposed the values of the seismic force reduction factor and the displacement amplification factor based on a combination of laboratory test results and numerical simulation of 14 autoclaved aerated concrete shear-wall specimens. Design displacement, ductility demand, equivalent viscous damping and capacity design concepts were widely investigated and the direct displacement-based seismic design of structures was assessed (Priestley *et al*. 2007). The results of nonlinear static analyses of representative RC frame buildings located in Turkey to obtain their capacity curves were presented. The parameters such as yield over-strength ratio, fundamental period, post elastic stiffness, yield and ultimate drift ratios, and yield base shear coefficient were obtained from the idealized capacity curves of representative set of buildings selected from existing RC buildings. The derived capacity curves were compared with other studies and HAZUS recommendations (Yakut 2008). A probabilistic methodology was proposed for the calibration of the behavior factor relating its value with two fundamental parameters, the displacement ductility capacity measured at a relevant location of the structure and the failure probability (Costa *et al*. 2010). An automatic procedure to evaluate the seismic capacity of existing RC regular buildings in terms of nonlinear strength C_s, capacity displacement C_d and period T was presented. Application and potentialities of the software were shown in an example for the generic building of a class (Verderame *et al*. 2010). The seismic safety of the regular framed buildings is studied, using static and dynamic non-linear analysis. The displacement ductility, overstrength and behavior factors were calculated and compared with the EC-8 prescribed ones. Finally, the fragility curves and damage probability matrices were computed (Vielma *et al*. 2011). Arslan (2012) determined how ductility values of both elements and load bearing systems vary as parameters related to the conditions specified in the codes change. The value of the curvature ductility was found to be 60 and 135 in the beam section and column section, respectively. Pushover analyses were applied to 540 different statuses of the sample four-story RC system, and the ratio variations and respective displacement (global) ductility of the frames were calculated. Zahid *et al*. (2013) investigated the overstrength factor of reinforced concrete frame designed according to EC2 and EC8 using pushover analysis. Regular and irregular frames in elevation with setback designed to gravity load only and designed to resist seismic load with medium ductility and high ductility class were considered in the study. It was found that, the geometry and ductility supply of the frames effect the overstrength factor. Zhou *et al*. (2014) examined the statistical relationship between the curvature ductility demands of columns and the global displacement ductility demands of five- and ten- story RC frames when subjected to earthquakes. The maximum global displacement ductility demands of the structure and the maximum curvature ductility demands of the columns were determined from pushover and time-history analysis. Carrillo *et al*. (2014) compared displacement ductility ratios of RC walls typically used in one- and two-story houses. Parameters for computing displacement ductility of low-rise RC walls, as the yield and maximum displacements, were discussed. An equation to estimate the displacement ductility capacity of a particular wall was proposed and displacement ductility capacity for code-based seismic design was recommended. Tawfik *et al*. 2014 presented an experimental study for the behavior and ductility factor of high strength reinforced concrete frames under lateral load. The experimental program was conducted on five frames. The results of the tests and the analysis of the obtained results were represented in terms of lateral load-displacement relationships, ductility, stiffness and energy dissipation. Taïeb and Sofiane (2014) evaluated ductility and overstrength in seismic design of reinforced concrete structures.

In this study, the yield base shear capacity, the yield displacement and the ultimate displacement capacity of three to eight story RC buildings, which constitute the major part of the existing building stock in Turkey, are derived from nonlinear static pushover analyses. Nonlinear static pushover analyses are performed by generating very detailed 3D models based on architectural and structural details of actual buildings. The capacity curve is approximated with a bilinear curve and characteristic points of capacity curves of each building are determined. The obtained values of the yield base shear coefficients and the displacement ductility factors are compared by those expected values given in different versions of Turkish Seismic Design Code and general evaluation of existing building stock in term of these parameters is presented.

2. The building inventory and database

The building inventory used in this study is composed of 30 three to eight story RC buildings which constitute the major part of the existing building stock in Izmir, the third biggest city of Turkey. These building are generally used for commercial or residential purposes and are selected from Konak and Karabaglar districts of city of Izmir. All of the selected existing buildings were designed according to 1975 version of Turkish Seismic Design Code (TSDC 1975). The major part of RC building stock in Turkey has similar design characteristics and the selected buildings reflect the construction practice in Turkey.

Numerical data provided by the research project, An Earthquake Damage Scenario and Earthquake Master Plan for the city of Izmir (1998), are also used in order to determine the extent of the present work. This research project was prepared mainly by the staff of the Department of Earthquake Engineering of the Kandilli Observatory and Earthquake Research Institute of Bogazici University in Istanbul in connection with the RADIUS project of the UN-IDNDR (International Decade for Natural Disaster Reduction) (Earthquake Risk Assessment for Istanbul Metropolitan Area 2002). The building inventory for the city of Izmir was assembled by expert civil engineers of Chamber of Civil Engineers of Izmir in 61 different areas of metropolitan municipality as a part of Earthquake Damage Scenario and Earthquake Master Plan for the city of Izmir (1998). The buildings used this study are selected from these areas.

The building inventory provides information about 190419 reinforced concrete buildings, 23362 buildings and 4043 other buildings (Earthquake Damage Scenario and Earthquake Master Plan for the city of Izmir 1998). The major part (87%) of existing buildings in the inventory is composed of RC buildings and that is why RC buildings are taken into consideration in the presented study. This percentage of RC buildings may be generalized for Turkey.

The building data used in this study is provided by a walkdown survey in four different locations of the pilot areas, which are assumed to represent the whole distinct. By this way, design characteristics and construction practices in the pilot area are reflected to the study. In walkdown survey many buildings are evaluated and address information of these buildings is taken. Block and section numbers of the buildings are determined by using the address information of the buildings as an input data for the computer program used in municipality. Architectural and reinforcement projects are provided from the archive of the municipality. Buildings which have similar design are eliminated and three to eight story 30 RC buildings are selected to be analyzed.

Table 1 General properties of the selected buildings

Story number	Building ID	Construction year	Total building height (m)	Materials
3	B3_1	1983	8.64	C14-S220
	B3_2	1990	8.50	C14-S220
	B3_3	1992	8.40	C16-S220
	B3_4	1995	8.40	C14-S220
	B3_5	1996	9.30	C14-S220
4	B4_1	1975	11.10	C14-S220
	B4_2	1981	11.70	C14-S220
	B4_3	1982	11.70	C14-S220
	B4_4	1984	11.40	C14-S220
	B4_5	1997	10.80	C16-S220
5	B5_1	1985	14.00	C16-S220
	B5_2	1989	14.30	C16-S220
	B5_3	1991	13.90	C14-S220
	B5_4	1991	13.50	C16-S220
	B5_5	1997	13.80	C14-S220
6	B6_1	1976	16.80	C14-S220
	B6_2	1985	15.80	C16-S220
	B6_3	1986	16.80	C14-S220
	B6_4	1995	16.30	C14-S220
	B6_5	1997	16.20	C18-S420
7	B7_1	1976	19.10	C16-S220
	B7_2	1977	18.55	C18-S220
	B7_3	1978	18.76	C18-S220
	B7_4	1994	18.90	C20-S420
	B7_5	1994	18.90	C20-S420
8	B8_1	1976	21.85	C18-S420
	B8_2	1978	21.75	C18-S420
	B8_3	1982	22.60	C14-S220
	B8_4	1982	21.80	C14-S220
	B8_5	1983	22.90	C18-S220

3. General characteristics of the selected buildings

Storey plans of each different floor of the selected buildings are drawn by using the data provided in architectural and reinforcement projects of the buildings. In cases of uncertainty in the projects, the data is provided by investigations on site. General properties of the selected buildings with numbers of stories ranges from three to eight are given in Table 1.

In building codes of Table 1, the first number represents the number of stories of the buildings and the second number stands for a building number. In material codes, the capital C stands for concrete and the following number represents the compressive strength of concrete material in MPa. Similarly, the letter S followed by a number representing the yield strength in MPa stands for reinforcement steel. Cross sectional dimensions of structural members and areas and configurations of longitudinal and transverse reinforcement are provided from reinforcement projects of the buildings. In this way, a valuable database, which reflects general design characteristics, engineering and construction practices in Turkey, is provided.

General characteristics of existing RC buildings in pilot areas, which are observed by the authors, are listed below.

- The structural system of 3-, 4- and 5-story buildings is moment resisting frame with monolithic beam-column connection..
- Shear wall-frame systems are not used in buildings which have number of stories less than 6.
- Although there exist moment resisting framed buildings ranging from 6 to 8 stories, shear wall-frame systems are widely used in these buildings.
- The value of distributed live load considered in structural design of the buildings is 2 kN/m^2, 3.5 kN/m^2 and 5 kN/m^2 for interior floors, stairs and exterior balconies, respectively.
- Values of 2.65 m, 2.70 m and 2.80 m are typical storey heights. The height of first storey is range up to 3.9 m in buildings which's first storey is used for commercial purposes.
- Typical slabs thickness is 10 cm, 11 cm and 12 cm for interior floors and 15 cm for balconies. Slabs of 13 cm and 14 cm thickness are rarely used in some buildings.
- Typical beam width is 25 cm and height is 50 cm in buildings constructed between the years 1975 and 1995. The beam width is generally 25 cm in buildings constructed after 1995.
- Column dimensions are range from 25/40 cm to different values depending on building story numbers and characteristics of the project.
- Typical diameter of stirrups used both in beams and columns is 8 mm. Typical spacing of stirrups is 10 cm and 20 cm for confinement zones and central zone of beams and columns.
- In some cases shallow beams are used in buildings which's first storey is used for commercial purposes.
- Column footings are generally used in 3-5 storey buildings. Besides, continuous footings and combined column and continuous footings are used.
- Continuous footings are typically used in 6 storey buildings. Although continuous footings are used in 7 and 8 storey buildings, raft foundations are typically used in these buildings.

The storey plan of B5_4 is given in Fig. 1.

4. Modeling approach and nonlinear modeling details

Three-dimensional analytical models of the selected RC buildings are generated and pushover analyses are performed by using these models. In modeling approach, geometric and material properties and reinforcement details are accepted to be in accordance with the existing project of the building. Beam-column joints are considered as infinitely rigid end zones. The floors are modeled with shell elements and idealized as rigid diaphragms. Shear walls are modeled as equivalent column elements having same section properties. To reveal the real behavior of shear walls under bending effect, rigid beams are used (Korkmaz *et al*. 2013). Three-dimensional analytical models of the buildings are created in the structural finite element software, SAP2000, Version 15.1.0 (Computers and Structures Inc. 2011).

Configurations of interior and exterior masonry infill walls are determined in accordance with the architectural project of the buildings. 3.80 kN/m^2 and 2.50 kN/m^2 distributed loads are assumed for 20 cm and 10 cm thick brick walls, respectively. Distributed gravity loads on the floors are assumed to be 1.50 kN/m^2 and distributed live loads are considered as indicated in design projects. Gravity loads of structural components are automatically taken into account by the software.

The story masses are assumed to be concentrated at the center mass of each story. Two horizontal degrees of freedom in perpendicular directions and a rotational degree of freedom with respect to the vertical axis passing through the mass center shall be considered at each storey. The floor masses are defined in accordance with the floor weights (the dead loads, G) plus 30% of the live loads, Q). Live load participation factor, n, is taken as 0.30 for residential buildings.

The initial effective stiffness values $(EI)_e$ of structural elements are reduced according to Turkish Seismic Design Code (2007) in order to account for cracking in sections during the inelastic response of buildings. For beams $(EI)_e$=0.40$(EI)_o$, and for columns and shear walls $(EI)_e$=0.40$(EI)_o$ if $N_D/(A_c f_{cm})\leq 0.10$ and $(EI)_e$=0.80$(EI)_o$ if $N_D/(A_c f_{cm})\geq 0.40$ are assumed, where A_c is cross section area of column or shear wall and f_{cm} is compression strength of concrete. Linear interpolation is applied for intermediate values of the axial compression force, N_D. Values of N_D are determined from gravity load analysis of the building considering seismic masses and using rigidities of uncracked section, $(EI)_o$.

The stress-strain relationships proposed by Mander *et al*. (1988) are implemented for unconfined and confined concrete. Section analyses are performed by the computer code XTRACT (2006). The stress-strain relation of unconfined and confined concrete having compression strength of 14 MPa shown in Fig. 2 is obtained by using a beam section with 20/50 cm dimensions and ϕ8/10 stirrup.

Reinforcement steel is modeled by parabolic strain hardening steel model, which is given in Turkish Seismic Design Code (2007). The stress-strain relation obtained by the software XTRACT (2006) for steel with tensile strength of 220 MPa is shown in Fig. 3.

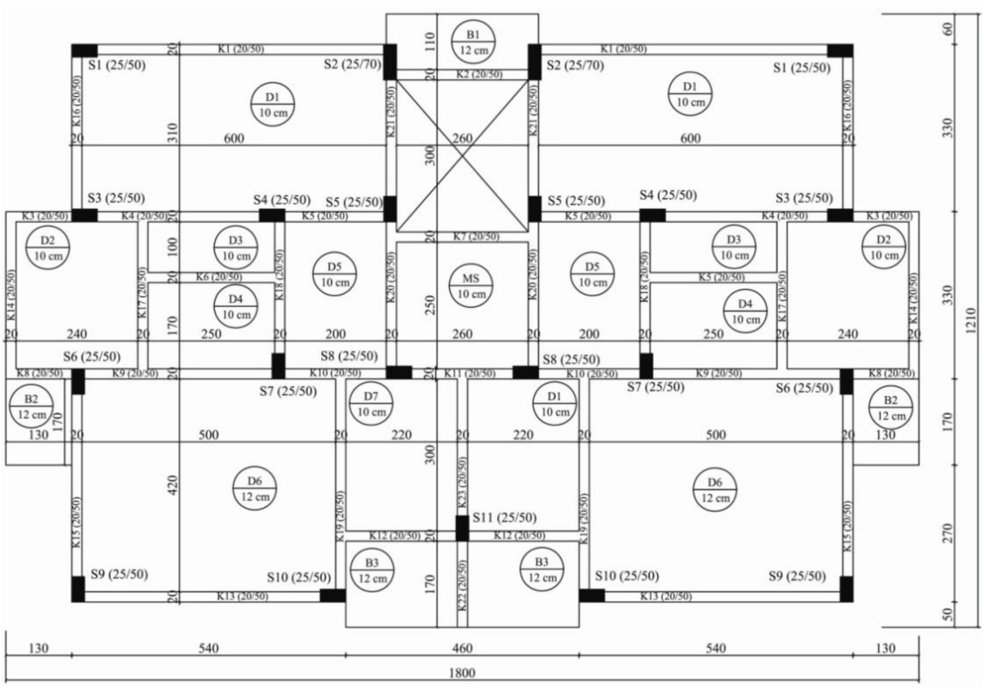

Fig. 1 Storey plan of B5_4

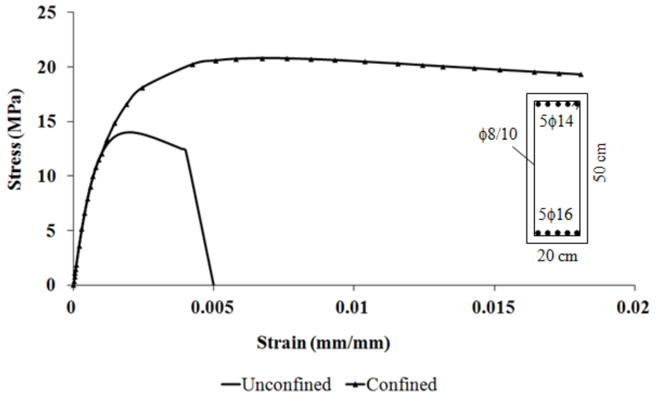

Fig. 2 Stress-strain relation for concrete

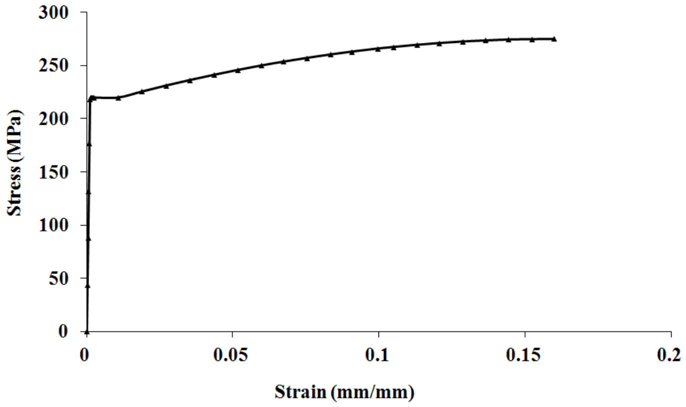

Fig. 3 Stress-strain relation for S220

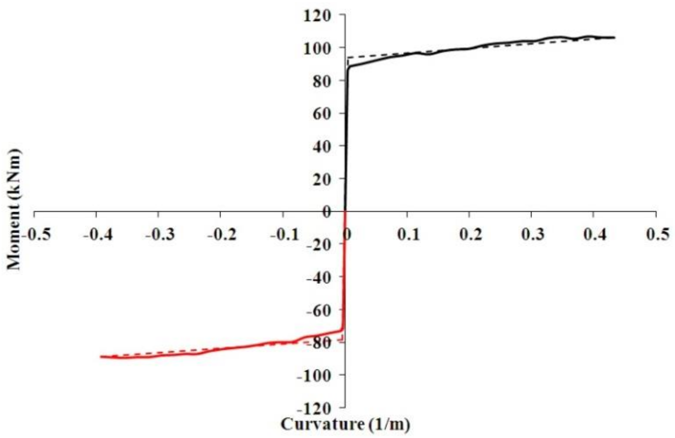

Fig. 4 Moment-curvature diagram and its bilinear representation

Material nonlinearity is idealized by adopting a lumped plasticity model, which provides an extensive practicability in engineering applications and corresponds to plastic hinge hypothesis in

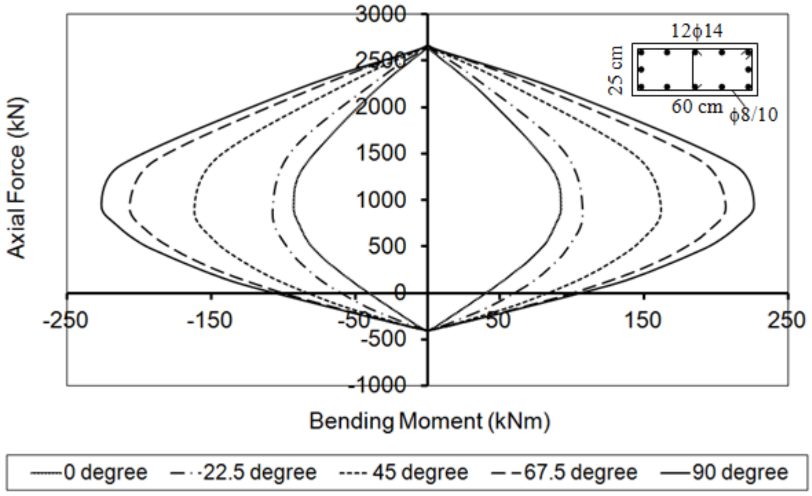

Fig. 5 Bending moment-axial force interaction diagram

pure bending. In this model, plastic deformations are assumed to be distributed uniformly along finite-length zones where plastic capacity of internal forces of structural elements is achieved. The length of plastic deformation zone, which is referred to as plastic hinge length (L_p), taken as the half of the section depth (h). Strain hardening effect is taken into consideration in order to determine internal force-plastic deformation relations of plastic hinges.

Beams and columns are modeled as nonlinear frame elements by defining plastic hinges at both ends of these elements. Moment-curvature analysis of sections is performed by using the cross section analysis program XTRACT (2006). Constant axial force of the columns is determined from gravity loads while axial force is assumed to be zero on the beams. Axial force of columns and shear walls are determined as the sum of the dead loads and 30% of the live loads on these elements (G+0.3Q). In order to define nonlinear behavior of beam, column and shear wall sections nearly 17500 number of moment-curvature are carried out. Positive and negative moment-curvature diagram and its bilinear representation of the beam section of Fig. 2 are shown in Fig. 4.

Column capacities are calculated from axial force-bending moment diagrams, which are also obtained by the computer code XTRACT (2006). Three dimensional axial force-bending moment interaction curves about horizontal axis, vertical axis and axes rotate by 22.5°, 45° and 67.5° about horizontal axis are created and used as input data for SAP2000 (Computers and Structures Inc. 2011). A typical bending moment-axial force interaction diagram is shown is Fig. 5.

Shear capacity of elements are calculated in accordance with TS-500 (2000) by considering shear capacities provided by concrete and shear reinforcement (stirrups). Flexural hinges (M3) are assigned for beams and interacting hinges (P-M2-M3) for columns.

Plastic rotation-moment relationships of plastic hinges, which are the required inputs for SAP2000 (Computers and Structures Inc. 2011), are obtained by multiplying plastic curvatures by the plastic hinge length. Plastic hinges are assigned at both ends of the beams and columns, and at bottom end of shear walls. Bottom end of a shear wall is a potential section where plastic hinges may be formed under lateral loads.

Table 2 Modal characteristics of the buildings

Story Number	Build. ID	X-Direction				Y-Direction			
		Uncracked		Cracked		Uncracked		Cracked	
		Period (sec)	Modal participating mass ratio	Period (sec)	Modal participating mass ratio	Period (sec)	Modal participating mass ratio	Period (sec)	Modal participating mass ratio
3	B3_1	0.432	0.6861	0.611	0.6971	0.332	0.7979	0.483	0.8121
	B3_2	0.335	0.8536	0.487	0.8464	0.416	0.6220	0.584	0.6197
	B3_3	0.477	0.6527	0.678	0.6344	0.433	0.8770	0.626	0.8611
	B3_4	0.369	0.6838	0.541	0.6707	0.318	0.8320	0.460	0.8250
	B3_5	0.271	0.8429	0.394	0.8541	0.545	0.9535	0.799	0.9531
4	B4_1	0.402	0.8821	0.583	0.8737	0.436	0.6163	0.619	0.5422
	B4_2	0.547	0.8988	0.765	0.8926	0.504	0.5531	0.692	0.4521
	B4_3	0.537	0.8259	0.756	0.8919	0.487	0.8895	0.689	0.8777
	B4_4	0.454	0.8083	0.651	0.8051	0.392	0.8150	0.569	0.8084
	B4_5	0.427	0.7942	0.611	0.7583	0.490	0.7962	0.713	0.7898
5	B5_1	0.310	0.7339	0.443	0.7383	0.462	0.8067	0.653	0.8041
	B5_2	0.541	0.8245	0.759	0.8117	0.593	0.6885	0.815	0.6709
	B5_3	0.547	0.7196	0.755	0.7344	0.450	0.7871	0.651	0.7732
	B5_4	0.526	0.5572	0.740	0.4598	0.527	0.8306	0.732	0.8188
	B5_5	0.489	0.8010	0.692	0.7817	0.414	0.4736	0.635	0.4596
6	B6_1	0.558	0.7927	0.779	0.7712	0.675	0.8033	0.925	0.7853
	B6_2	0.746	0.5684	0.970	0.5974	0.553	0.6431	0.769	0.6184
	B6_3	0.660	0.5572	0.897	0.5246	0.702	0.7890	0.946	0.7790
	B6_4	0.789	0.7940	1.031	0.7772	0.598	0.6850	0.776	0.6371
	B6_5	0.541	0.6257	0.755	0.6376	0.370	0.7674	0.486	0.7758
7	B7_1	0.618	0.7987	0.870	0.7953	0.506	0.6098	0.680	0.5251
	B7_2	0.678	0.5991	0.932	0.5804	0.556	0.7372	0.784	0.7481
	B7_3	0.412	0.6178	0.569	0.5338	0.589	0.6798	0.843	0.6707
	B7_4	0.634	0.5902	0.861	0.5747	0.715	0.7850	0.970	0.7609
	B7_5	0.760	0.7814	1.041	0.7766	0.608	0.7753	0.812	0.7687
8	B8_1	0.826	0.4341	1.114	0.4059	0.652	0.4318	0.859	0.4791
	B8_2	1.003	0.6801	1.378	0.6579	0.578	0.7971	0.805	0.7885
	B8_3	0.797	0.6770	1.072	0.6657	0.693	0.4547	0.943	0.4148
	B8_4	0.863	0.6015	1.181	0.5881	0.762	0.7909	1.050	0.7819
	B8_5	0.732	0.6351	0.992	0.7897	0.860	0.6208	1.174	0.6007

5. Modal properties and nonlinear static analysis of buildings

Modal parameters for both perpendicular directions of the buildings are determined from eigen vector analysis of each building in SAP2000 (Computers and Structures Inc. 2011) environment. Natural periods and modal participating mass rations of undamped free-vibration of the selected buildings are given in Table 2. The above mentioned modal parameters are determined by using both uncracked and cracked sections.

Pushover analyses are performed using three dimensional models of buildings which are created in SAP2000 (Computers and Structures Inc. 2011). Two different lateral load patterns are used in pushover analyses considering modal participating mass ratios of the buildings. A vertical distribution proportional to the shape of the fundamental mode in the direction under consideration is used when more than 70% of the total mass participates in this fundamental mode (TSDC 2007). The most current nonlinear static procedures use lateral load patterns based on the first mode, which are adequate for structures whose response is controlled by the fundamental vibration mode. For structures with significant higher mode response, such as plan-asymmetric structures whose three-dimensional seismic response in the inelastic range is very complex, the contribution of all significant modes of vibration should be considered (Gupta and Kunnath 2000, Chopra and Goel 2002, Aydinoglu 2003). A uniform distribution of lateral forces is used when less than 70% of the total mass participates in the fundamental mode in the direction under consideration (Korkmaz 2005).

Nonlinear static analyses considering only seismic masses are performed before pushover analyses. The results of these analyses are considered as initial conditions of pushover analyses and pushover analyses are carried out by using two different lateral load patterns. The obtained capacity curves of 3-, 4-, 5-, 6-, 7-, and 8- story buildings for both directions into consideration are sketched in Figs. 6-11, respectively.

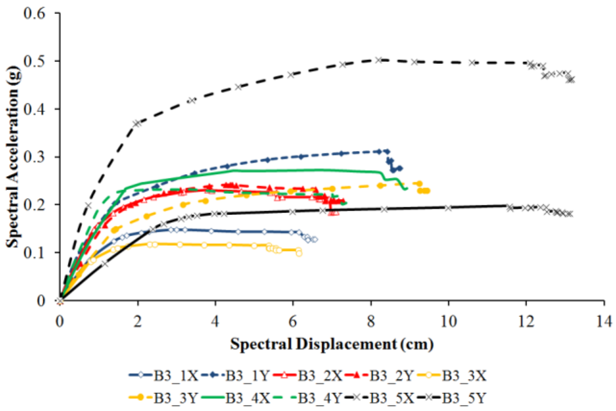

Fig. 6 Capacity curves of 3-story buildings

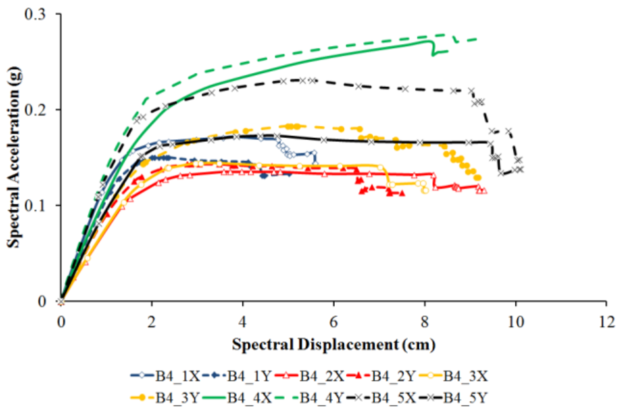

Fig. 7 Capacity curves of 4-story buildings

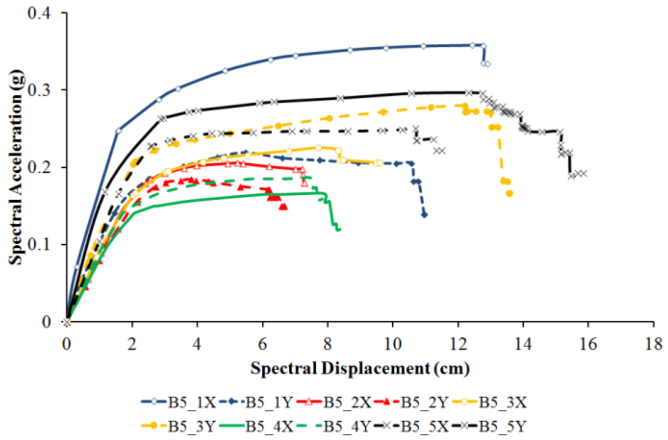

Fig. 8 Capacity curves of 5-story buildings

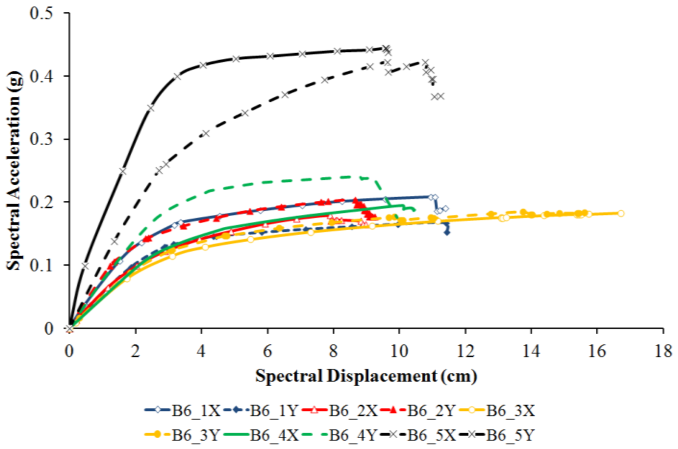

Fig. 9 Capacity curves of 6-story buildings

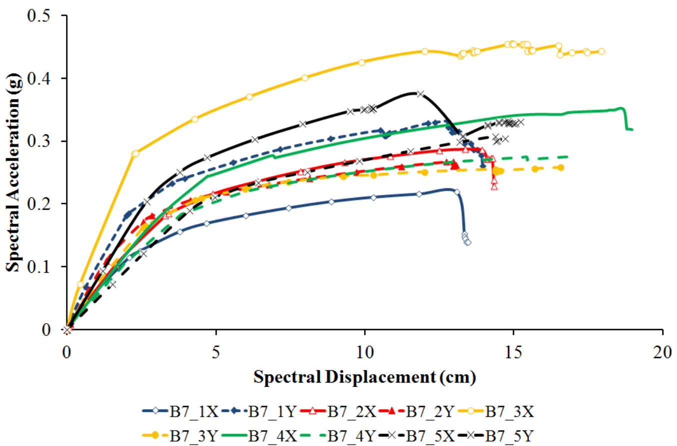

Fig. 10 Capacity curves of 7-story buildings

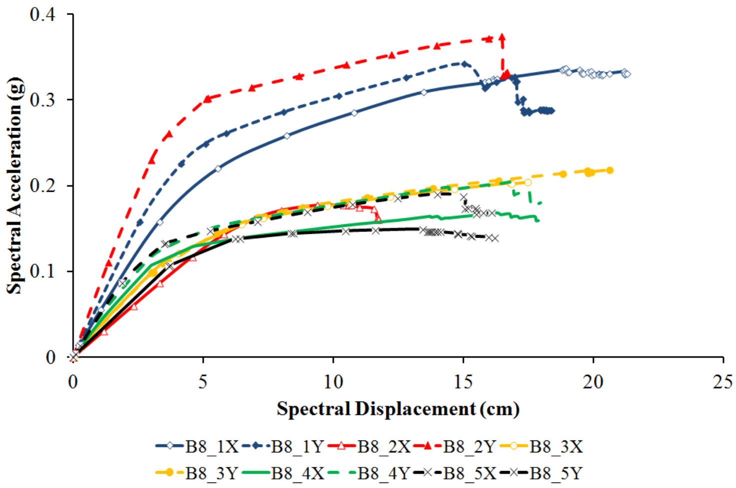

Fig. 11 Capacity curves of 8-story buildings

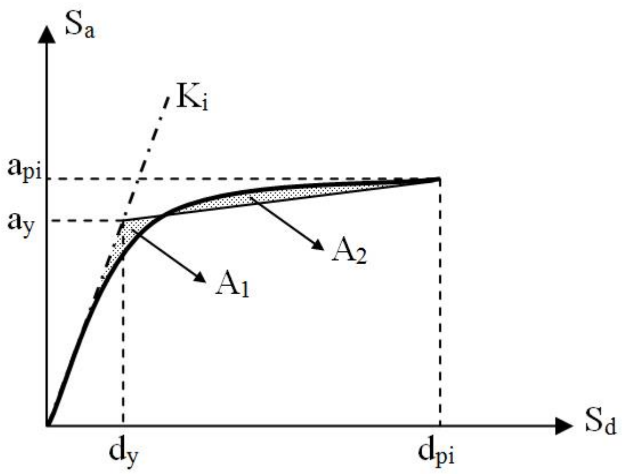

Fig. 12 Bilinear representation of capacity spectrum (ATC-40 1996)

6. Idealization of capacity curves; displacement ductility factor and base shear coefficients calculations

Capacity curve of the building needs to be idealized in order to determine the yield base shear capacity, the yield displacement and the ultimate displacement. In literature there is no universal consensus on how to idealize the capacity curve of the building. The idealization concept was first used for idealization of force-deformation relationships of RC elements when calculating ductility factors and various estimations were made by different investigators (Priestley and Park 1987, Park 1988, Paulay and Priestley 1992, Paulay 2002, Faella *et al*. 2004, Sullivan *et al*. 2004).

The idealization of force-displacement relationship of RC structures is analogous to idealization of force-deformation relationships of RC members. Two different methods that could be used to estimate yield forces and yield displacements from bilinear representation of capacity

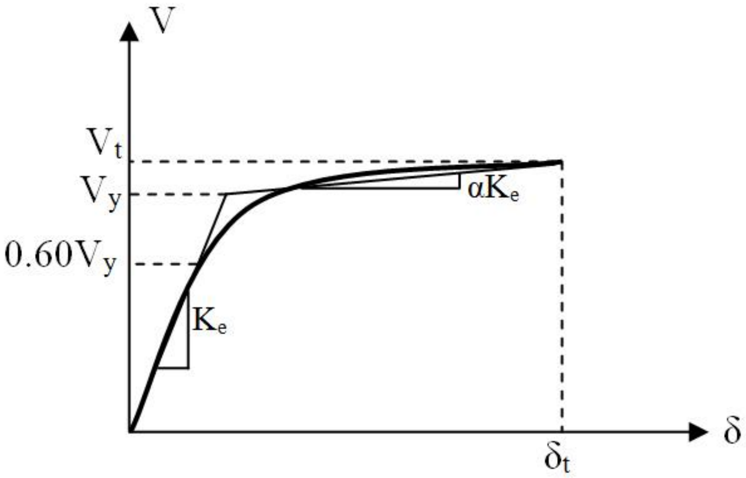

Fig. 13 Bilinear representation of pushover curve (FEMA-356 2000)

curve were introduced in ATC-19 (1995). Another idealization technique of capacity curve was given in ATC-40 (1996) as a part of the Capacity Spectrum Method. In this method, construction of bilinear representation of capacity spectrum is obtained by drawing one line up from the origin at the initial stiffness and a second line back from the trial performance point such that to have equal energy associated with the capacity spectrum and its bilinear representation (Fig. 12).

In FEMA-356 (2000), the nonlinear base shear and displacement relationship of the control node was replaced with a bilinear relationship with initial slope K_e and post-yield slope α to calculate the effective lateral stiffness, K_e, and effective yield strength, V_y, of the building. The effective lateral stiffness was taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure (Fig. 13).

In this study, the capacity curve of each building is approximated with a bilinear curve similar to given in FEMA-356 (2000). The base shear coefficient is obtained as the ratio of yield base shear capacity (V_y) to the building weight (W). The yield displacement (δ_y), which corresponds to significant yielding of the system, is taken as the displacement value at the intersection point of idealized curves. The ultimate displacement capacity (δ_u) is determined as the state at which the building reaches its deformation capacity. The obtained values of base shear coefficients for both directions into consideration are given in Table 3. Also, the smallest values of ratio of yield displacement (δ_y) to total building height (H), the ratio of ultimate displacement capacity of buildings (δ_u/H) and displacement ductility factors (δ_u/δ_y) are listed in Table 3.

7. Conclusions

In the presented study, the yield base shear capacity, the yield displacement and the ultimate displacement capacity of three to eight story existing RC buildings, which were designed according to 1975 version of Turkish Seismic Design Code, are determined from capacity curves of the buildings. Important findings representing general characteristics of the existing RC building stock are obtained.

Table 3 Statistics of characteristic points of pushover curves

Building ID	$V_{y,x}/W$	$V_{y,y}/W$	δ_y/H	δ_u/H	δ_u/δ_y
B3_1	0.132	0.242	0.002281	0.008533	3.74090
B3_2	0.204	0.211	0.002090	0.009904	4.73876
B3_3	0.109	0.187	0.001556	0.007447	4.78599
B3_4	0.233	0.208	0.002092	0.010458	4.99904
B3_5	0.369	0.170	0.004234	0.015842	3.74162
Mean values	0.209	0.204	0.002451	0.010437	4.40126
B4_1	0.151	0.132	0.001337	0.004342	3.24757
B4_2	0.118	0.124	0.001636	0006549	4.00306
B4_3	0.129	0.158	0.002168	0.007273	3.35470
B4_4	0.190	0.217	0.002837	0.010110	3.56362
B4_5	0.193	0.151	0.002612	0.010645	4.07542
Mean values	0.156	0.156	0.002118	0.007784	3.64887
B5_1	0.260	0.178	0.002336	0.009652	4.13185
B5_2	0.169	0.144	0.002851	0.006594	2.31287
B5_3	0.180	0.211	0.002632	0.008759	3.32789
B5_4	0.148	0.154	0.002204	0.007205	3.26906
B5_5	0.205	0.252	0.002590	0.010188	3.93359
Mean values	0.192	0.188	0.002523	0.008480	3.39505
B6_1	0.145	0.129	0.002562	0.009152	3.57221
B6_2	0.108	0.147	0.002028	0.006716	3.31164
B6_3	0.129	0.125	0.002745	0.011270	4.10565
B6_4	0.136	0.172	0.001730	0.004194	2.42428
B6_5	0.271	0.384	0.002667	0.008323	3.12073
Mean values	0.158	0.191	0.002346	0.007931	3.30690
B7_1	0.149	0.218	0.002709	0.009237	3.40975
B7_2	0.203	0.160	0.001875	0.009336	4.97920
B7_3	0.307	0.179	0.002264	0.010966	4.84364
B7_4	0.254	0.185	0.003481	0.011608	3.33467
B7_5	0.191	0.251	0.004023	0.010741	2.66990
Mean values	0.221	0.199	0.002870	0.010378	3.84743
B8_1	0.218	0.230	0.002569	0.009793	3.81199
B8_2	0.121	0.291	0.004684	0.008974	1.91588
B8_3	0.135	0.142	0.003800	0.010859	2.85763
B8_4	0.112	0.146	0.002184	0.010220	4.67949
B8_5	0.150	0.117	0.001579	0.006503	4.11843
Mean values	0.147	0.185	0.002963	0.009270	3.47668

The obtained mean values of lateral displacement ductility factor are 3.78 and 3.28 for reinforced concrete moment resisting frame buildings and shear wall-frame buildings, respectively. These values correspond to quite low displacement ductility factors. Using the values of Turkish Seismic Design Code, where overstrength factor is 1.5 and structural system behavior factor is 8

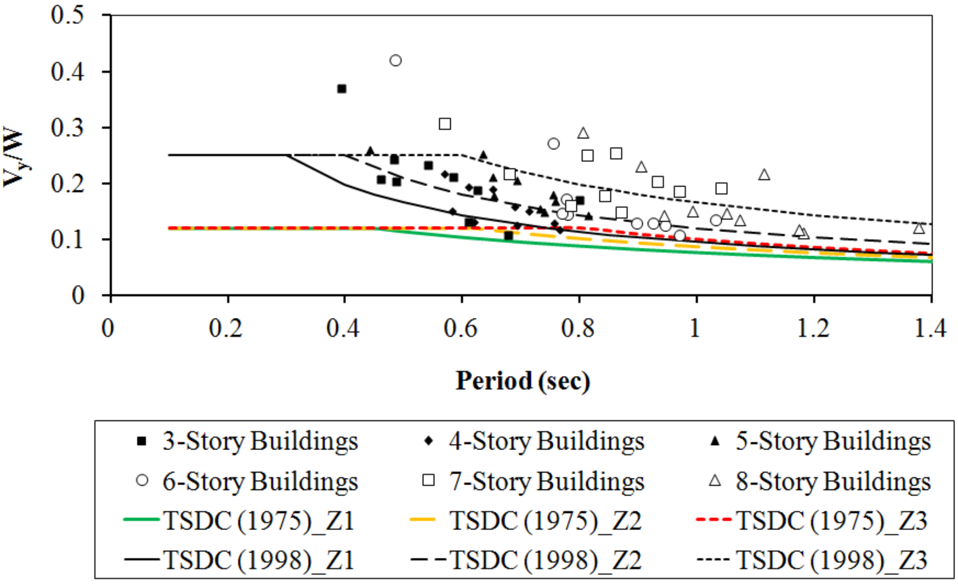

Fig. 14 Variation of base shear coefficient with effective period of the buildings

for buildings in which seismic loads are fully resisted by high ductile frames, the expected value of displacement ductility factor is about 5.3 for moment resisting frame buildings. The expected value of displacement ductility factor is about 4.6 for RC buildings, in which seismic loads are resisted by ductile frames and shear walls. The ratio of ultimate displacement capacity of buildings to total building height (δ_u/H) is about 1% for all buildings. The values of ratio of yield displacement to total building height (δ_y/H), is between 0.2% and 0.3%.

In order to evaluate lateral strength characteristics of the buildings, the base shear coefficient is obtained as the ratio of yield base shear capacity (V_y) to the building weight (W). The variation of yield base shear coefficient with effective period for the buildings is shown in Fig. 14. The spectral variations of yield base shear coefficient of 1975 and 1998 versions of Turkish Seismic Design Codes for different local site classes (Z1, Z2 and Z3) are also plotted in Fig. 14. When compared with code requirements, quite high values of yield base shear coefficient are obtained. Some of these buildings, which were designed according to 1975 version of Turkish Seismic Design Code and expected to conform to the provisions of this code, also provide the requirements of 1998 version of Turkish Seismic Design Code, which have high seismic design provisions.

The main findings of this study outline that, although typical mid-rise RC buildings in Turkey have sufficient lateral strength capacity, their deformation capacity is considerably low. Damage associated with insufficient deformation capacity of structural elements and building may be expected. If strengthening is required for these types of buildings, strengthening techniques to improve deformations capacities of elements should be used.

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