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Seismic performance evaluation of school buildings in Turkey

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Abstract. This study evaluates seismic performance of the school buildings with the selected template designs in Turkey considering nonlinear behavior of reinforced concrete components. Six school buildings with template designs were selected to represent major percentage of school buildings in medium-size cities located in high seismic region of Turkey. Selection of template designed buildings and material properties were based on field investigation on government owned school buildings in several cities in western part of Turkey. Capacity curves of investigated buildings were determined by pushover analyses conducted in two principal directions. The inelastic dynamic characteristics were represented by equivalent single-degree-of-freedom (SDOF) systems and their seismic displacement demands were calculated under selected ground motions. Seismic performance evaluation was carried out in accordance with recently published Turkish Earthquake Code that has similarities with FEMA-356 guidelines. Reasons of building damages in past earthquakes are examined using the results of performance assessment of investigated buildings. The effects of material quality on seismic performance of school buildings were investigated. The detailed examination of capacity curves and performance evaluation identified deficiencies and possible solutions for template designs.

Keywords: Seismic performance evaluation; pushover analysis; reinforced concrete structure; school buildings.

1. Introduction

Earthquake prone countries need earthquake-resistant school buildings. Closure of schools due to earthquake damage may result in community problems; education is hampered, community life is disrupted and emergency shelters are unavailable. However, the most important reason for earthquake-resistant schools is the safety of school children and teachers.

Recent devastating earthquakes in Turkey and in other countries such as Algeria, India, Iran, and Morocco in the world have emphasized inadequate seismic performance of school buildings (OECD 2004). In literature, there are many studies related to performance of school buildings in past

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Table 1 School damages in Turkey during recent earthquakes

Earthquake	Date (dd/mm/yy)	Magnitude	# of lightly and moderately damaged schools	# of heavily damaged schools
Erzincan	13.03.1992	$M_s = 6.8$	34	9
Kocaeli	17.08.1999	$M_s = 7.4$	381*	43*
Kocaeli	17.08.1999	$M_s = 7.4$	807**	13**
Bingol	01.05.2003	$M_w = 6.4$	13	13

*Schools in Kocaeli and surrounding area; **Schools in Istanbul

earthquakes (Ozcebe *et al.* 2004, Hassan and Sozen 1997, EERI Special Earthquake Report 2003, Eshghi and Naserasadi 2003, Yeh *et al.* 2006). Many school buildings are affected by destructive earthquakes due to poor quality of construction, poor workmanship, and lack of maintenance. A summary of school buildings damaged during recent earthquakes in Turkey is given in Table 1.

Following damages observed in past earthquakes (i.e., 1999 Kocaeli, 1999 Duzce and 2003 Bingol earthquakes in Turkey, 2001 Bhuj earthquake in India, 2003 Boumers earthquake in Algeria, 2003 Bam earthquake in Iran, and 2004 Al Hoceima earthquake in Morocco), there have been significant efforts to reduce seismic hazards in school buildings in Turkey and in many other countries (OECD 2004).

Seismic safety of public buildings has been questioned in the wake of 1999 Kocaeli, 1999 Duzce and 2003 Bingol earthquakes in Turkey because there was a widespread conviction that these buildings experienced considerable damage compared to privately-owned properties. Although randomly sampled statistics on the number of damaged structures may not directly support this perception, it is worth noting that a significant number of the reinforced concrete buildings damaged during 2003 Bingol earthquake were government buildings such as schools, dormitories, and state buildings (Dogangun 2004, EERI Special Earthquake Report 2003).

In Turkey, template designs developed by the General Directorate of Construction Affairs are used for many of the buildings intended for governmental services (administrative centers, health clinics, hospitals, schools etc.) as common practice to save on architectural fees and ensure quality control. Hence, there are standard buildings all over the country for ten-classroom schools, or 120-bed hospitals. Majority of existing public buildings were constructed per 1975 Turkish Earthquake Code (TEC-1975) that was in use until the modern code became in force in 1998 (TEC-1998). The recently published earthquake code includes small modifications in the 1998 code and a new chapter for seismic evaluation of existing buildings (TEC-2007).

This study aims to evaluate seismic performance of the school buildings constructed per pre-modern seismic code (TEC-1975) in Turkey considering nonlinear behavior of reinforced concrete components. Six school buildings with template designs were selected to represent major percentage of school buildings in medium-size cities located in high seismic region of Turkey. Selection of template designed buildings and material properties were based on field investigation on government owned school buildings in several cities in western part of Turkey. Capacity curves of investigated buildings were determined by pushover analyses conducted in two principal directions. The inelastic dynamic characteristics were represented by equivalent single-degree-of-freedom (SDOF) systems and their seismic displacement demands were calculated under selected ground motions. Seismic performance evaluation was carried out in accordance with recently published Turkish Earthquake Code-2007 (TEC-2007) that has similarities with FEMA-356 (2000) guidelines. Reasons of building damages in past earthquakes are examined using the results of performance

Table 2 Statistics of template design for 108 schools in Denizli

Template Design	# of Schools
TD-10370	47
TD-10419	25
TD-735	8
TD-10816	4

assessment of investigated buildings. Further details about the investigated buildings and their capacity evaluation can be found in Bilgin (2007) and Inel *et al.* (2008a).

1.1 Description of structures

A field survey was carried out in Denizli city to select the most common template designs in school buildings. Being an important industrial, tourism, and export center, Denizli represents a medium-size city in a seismically active part of Turkey. According to field survey, there were 161 buildings of 108 school complexes. Table 2 shows statistics for template designs, indicating that the most common templates are TD-10370, TD-10419, and TD-735 as primary school and TD-10816 as high school buildings. The TD-10419 template exists in both 4- and 5-story forms. The TD-735 comprises three buildings separated with expansion joints and two of them are selected, excluding the building used as entrance and exit with relatively small floor area. Hence, six RC buildings from four template designs were selected to represent major portion of school buildings in seismically active medium-size city based on the field survey; TD-10370, TD-10419 (4-story), TD-101419 (5-story), TD-735.A, TD-735.B and TD-10816 templates.

The selected school buildings are reinforced concrete moment resisting frame with shear walls (RC dual system) structures in both longitudinal and transverse directions except that TD-10419 template is reinforced concrete moment resisting frame structure in longitudinal direction while it is RC dual system in transverse direction. Table 3 lists summary of buildings including purpose of use, number of stories, number of classrooms, structural frame type, shear wall area normalized by total floor area of the building in both longitudinal and transverse and typical beam dimensions.

Table 3 Summary of the selected template designs

Properties		Template Design ID					
		TD-10370	TD-10419 (4-story)	TD-10419 (5-story)	TD-735.A	TD-735.B	TD-10816
Purpose of Use		Primary School	Primary & High School	Primary & High School	Primary & High School	Primary & High School	High School
Floor Area (m^2)		322	613	613	250	390	890
# of Stories		3	4	5	4	4	5
# of Classrooms		5	12	18	16		24
Structural Type	Long.	RC Dual	RC Frame	RC Frame	RC Dual	RC Dual	RC Dual
	Trans.	RC Dual	RC Dual	RC Dual	RC Dual	RC Dual	RC Dual
Shear wall area (% of building area)	Long.	0.37	---	---	0.23	0.13	0.10
	Trans.	0.26	0.38	0.30	0.37	0.25	0.20
Typical beam dimensions (mm)		250 × 500 300 × 700	300 × 800 400 × 800	300 × 800 400 × 800	300 × 600	300 × 600	300 × 700

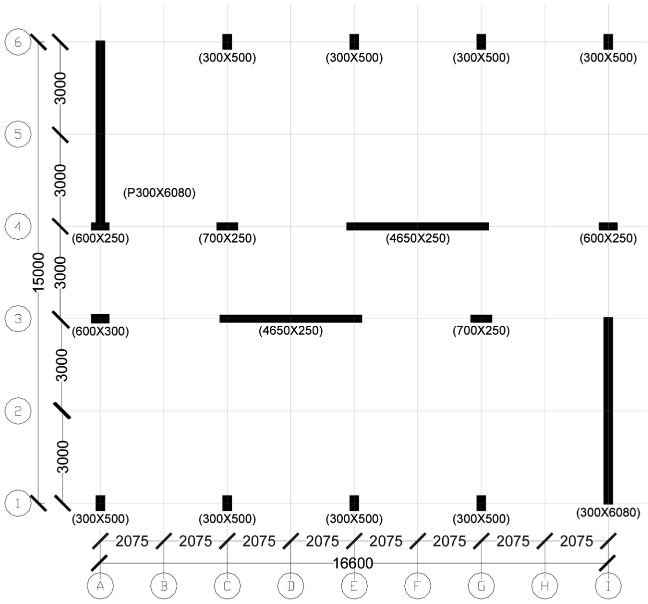

Fig. 1 Structural floor plan view of the TD-735.A building

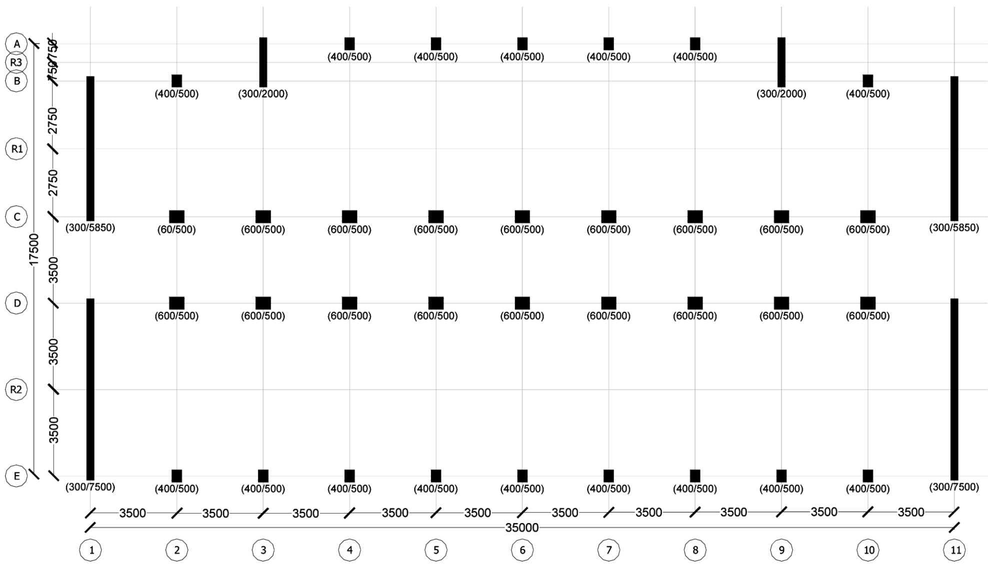

Fig. 2 Structural floor plan view of the TD-10419 building (dimensions in mm)

Representative plan views of two school buildings for the ground story are provided in Figs. 1 and 2, TD-735.A has shear walls in both longitudinal and transverse directions while TD-10419 has shear walls only in transverse direction.

2. Material Properties

As-built material properties determined from field investigation and experimental work were taken into account for nonlinear analysis of the selected school buildings. As aforementioned many of the buildings intended for governmental services (administrative centers, health clinics, hospitals, schools etc.) have similar construction procedure supervised by General Directorate of Construction Affairs. Material properties considered in this study were determined based on field study on 102 school buildings located in Denizli. Inel *et al.* (2008b) evaluated concrete strength of existing public buildings (i.e. schools, hospitals, governmental service buildings) located in three different cities based on core sampling and laboratory testing. Their work provides a global view on in-situ concrete strength of existing public building while current study only concentrates on schools.

Concrete strength of each building was determined using both nondestructive and destructive methods. Schmidt hammer was used as nondestructive method while core sampling and laboratory testing was carried out as destructive method. It should be noted that testing was carried out on lateral load resisting components (FEMA-356 2000). The aim for use of nondestructive method was to increase the number of sampling in each building. Randomly selected components for core sampling were identified to meet with minimum requirements of standards or prestandards (FEMA-356) for number of tests. First, Schmidt hammer test was applied on the selected components and core samples are taken from at least one third of them in order to correlate with Schmidt hammer tests. Cores of 53- or 64-mm diameter and different lengths were extracted.

The core samples were subjected to uniaxial compression in laboratory. The results were converted to compressive strength of standard cylinder (150 mm $\times$ 300 mm) using correction factor for the effect of length to diameter ratio (l/d). Schmidt hammer rebound readings and standard cylinder compressive strength values of core samples were evaluated to obtain a correlation; a typical linear equation of $y = ax + b$ was aimed for simplicity. Using the obtained correlation, compressive strength of all components in each building was determined. It should be kept in mind that since correlation between rebound readings and core strengths was used only in the related building, the same rebound readings in two different buildings may correspond to different compressive strength of sample cores.

The aforementioned method for determining compressive strength of a building was applied to 102 school buildings. Total of randomly selected 1040 core samples was extracted for laboratory testing while Schmidt hammer test was performed on 3256 components.

Once standard cylinder compressive strength of each component was determined in each building, the average and coefficient of variation (COV) values were obtained. Table 4 lists number of components for core sampling and Schmidt hammer reading, mean, standard deviation, and expected concrete strength values of each building.

The COV values range between 0.06 and 0.30. According to FEMA-356, the mean strength is allowed to be used as the expected strength in the analysis if COV in test results is less than 14%. Otherwise, the expected strength shall not exceed the mean less one standard deviation. Due to higher variation in the strength values observed in Table 4, the expected compressive concrete strength of each building was determined as mean less one standard deviation Eq. (1).

$$f_{c,\exp} = f_{c,mean} - \sigma \tag{1}$$

The expected concrete strength ranges between 6.2 and 27.5 MPa as plotted in Fig. 3. The figure indicates that the concrete strength ranges between 10 and 16 MPa for the most of buildings. Hence, three strength values-10, 13, and 16 MPa- were considered to represent typical concrete strength values of existing school buildings constructed per pre-modern code. All buildings of this study were constructed per the pre-modern code and their specified concrete strength is either 14

Table 4 Expected concrete strength of selected buildings

Building ID	Construction year	Core samples		Core & Schmidt hammer samples			Building
		# of samples	$f_{c,mean}$ (MPa)	# of samples	$f_{c,mean}$ (MPa)	σ (MPa)	$f_{c,expected}$ (MPa)
ABIO	1991	10	9.8	27	10.0	1.1	8.9
AEML-A	1979	24	12.7	70	13.6	3.0	10.6
AEML-E	1979	4	22.1	11	21.5	2.8	18.7
AEML-KSS	1979	14	10.7	50	11.2	2.5	8.7
AEML-O	1979	26	10.7	91	10.6	1.1	9.5
AIHL	1996	18	15.9	60	16.3	2.7	13.6
AIO	1997	6	10.6	16	9.8	1.4	8.4
AL-A	1983	22	17.8	72	17.6	3.3	14.3
AL-B	1983	8	15.2	18	15.9	2.7	13.2
AL-KSS	1983	10	20.1	25	20.2	3.1	17.1
ANEIOO	1997	13	17.1	38	19.1	3.5	15.8
ASUIO	1991	7	9.6	26	11.1	2.6	8.5
ATML-A	1991	8	12.0	28	12.4	2.4	10.0
ATML-B	1991	7	11.3	31	11.2	0.8	10.4
ATML-C I	1991	8	11.8	25	12.9	2.5	10.4
ATML-C II	1991	4	8.6	24	9.3	1.4	7.9
ATML-D	1991	12	13.2	35	13.2	1.8	11.4
AYIO	1994	13	21.3	33	21.0	2.0	19.0
BMKIOO.	1988	11	8.4	24	9.1	2.5	6.6
BSM	1990	15	13.8	41	13.1	2.7	10.4
CBIO	1985	17	13.0	59	12.7	3.3	9.4
CCIO	1990	8	10.3	32	11.1	1.7	9.4
CGIO	1984	12	11.0	34	11.7	2.0	9.7
CGIO-KSS	1984	7	10.4	22	11.4	2.8	8.6
CL-A	1975	18	12.3	54	12.0	2.2	9.8
CL-B	1973	7	9.1	21	11.9	2.0	9.9
CL-KSS	1990	6	20.0	15	19.8	2.6	17.2
CL-PB	1973	13	16.2	31	16.2	1.3	14.9
DAL-A	1987	6	12.4	15	12.2	1.0	11.2
DAL-B	1987	18	10.7	47	11.3	1.8	9.5
DAL-C	1987	5	9.1	10	10.3	2.1	8.2
DALP.A	1990	12	9.6	27	10.5	1.9	8.6
DALP.A	1990	12	9.7	28	9.9	1.4	8.5
DL-A	1970	28	18.6	76	18.9	3.3	15.6
DL-B	1970	12	9.6	33	9.7	1.3	8.4
DL-C	1991	22	13.4	51	12.3	2.7	9.6
EIOO	1992	9	8.0	21	8.2	1.2	7.0
FAAO-A	1996	6	16.0	16	16.2	2.0	14.2
FAAO-B	1996	5	16.7	15	17.8	2.0	15.8
FAAO-C	1996	3	16.2	8	17.2	1.0	16.2

Table 4 Expected concrete strength of selected buildings (Cont'd)

Building ID	Construction year	Core samples		Core & Schmidt hammer samples			Building
		# of samples	$f_{c,mean}$ (MPa)	# of samples	$f_{c,mean}$ (MPa)	σ (MPa)	$f_{c,expected}$ (MPa)
FAAO-D	1996	10	17.7	18	17.8	2.0	15.8
FIO	1982	15	13.8	45	13.7	1.3	12.4
FIO-KSS	1982	5	12.9	20	17.4	3.4	14.0
GEM	1986	8	11.1	331	11.2	1.8	9.4
GIO	1992	7	20.1	23	18.3	3.1	15.2
GIOO	1995	13	12.6	30	12.7	2.8	10.1
GUM	1995	19	13.1	49	12.7	1.2	11.5
HEM	1990	15	17.4	42	17.4	2.5	14.9
HIO-A	1979	6	8.3	17	11.0	2.2	8.8
HIO-B	1979	6	12.6	16	11.7	1.8	9.9
HIO-C	1979	9	12.9	21	12.0	2.0	10.0
KKL-A	1995	11	14.8	30	16.2	3.4	12.8
KKL-B	1995	5	15.0	23	14.4	4.0	10.4
KKL-C	1995	12	18.3	37	18.2	3.2	15.0
KML-A	1958	7	16.0	17	15.0	2.3	12.7
KML-B	1958	7	14.9	14	15.2	3.0	12.2
KML-C	1958	12	15.8	37	14.8	2.4	12.4
KML-I	1983	7	10.9	21	12.8	2.7	10.1
KML-II	2004	4	25.6	12	25.6	2.0	23.6
LEIO	1993	9	21.2	27	22.5	6.3	16.2
LOIO	1984	10	8.8	18	9.0	1.2	7.8
MAEL-A	1987	7	14.4	26	14.4	2.1	12.3
MAEL-B	1987	6	11.9	15	13.4	4.0	9.4
MAEL-C	1987	13	13.1	46	13.8	3.0	10.8
MBIO-A	1990	7	13.4	20	12.9	2.2	10.7
MBIO-B	1990	8	12.2	23	13.5	2.1	11.4
MIOO-A	1962	8	14.6	15	14.9	1.5	13.4
MIOO-B	1962	11	15.1	20	15.1	2.1	13.0
OYYDIO	1996	11	15.2	31	15.3	2.1	13.2
PAUEF-A	1960	10	18.8	28	23.0	4.9	18.1
PAUEF-B	1960	10	19.2	24	21.4	3.4	18.0
PAUHMYO	1996	10	20.8	17	21.3	2.6	18.7
PAUMF-A	1970	10	13.1	23	13.5	2.9	10.6
PAUMF-B	1970	12	13.3	22	14.2	2.7	11.5
RAM	1990	13	9.8	38	10.0	1.9	8.1
SAUIO	1999	8	9.5	22	10.4	1.9	8.5
SD	1990	6	10.8	23	11.3	1.2	10.1
SKIO-A	1966	7	10.7	16	12.4	2.4	10.0
SKIO-B	1966	13	14.5	26	13.7	3.3	10.4
TEVAL-A	1998	19	29.9	54	29.9	2.6	27.3

Table 4 Expected concrete strength of selected buildings (Cont'd)

Building ID	Construction year	Core samples		Core & Schmidt hammer samples			Building
		# of samples	$f_{c,mean}$ (MPa)	# of samples	$f_{c,mean}$ (MPa)	σ (MPa)	$f_{c,expected}$ (MPa)
TEVAL-B	1998	11	30.3	31	29.7	2.2	27.5
TEVAL-C	1998	5	28.3	12	30.1	2.9	27.2
TML.A	1996	12	15.3	33	15.8	2.4	13.4
TML.B	1980	11	15.9	29	16.1	3.1	13.0
TML.C	1973	5	12.5	19	11.8	1.9	9.9
TML.D	1973	7	13.8	20	13.0	2.2	10.8
UIO	1990	5	9.7	15	10.3	1.8	8.5
VNBEM-A	1990	12	13.8	30	14.3	2.3	12.0
VNBEM-B & C	1990	13	12.4	67	13.0	2.2	10.8
YBEML-AI	1943	5	16.7	10	18.7	1.7	17.0
YBEML-EA	1973	5	20.6	10	20.0	4.2	15.8
YBEML-KSS	1966	5	21.8	19	21.6	3.1	18.5
YBEML-M	1973	24	19.8	63	19.3	2.0	17.3
YBEML-OB	1966	6	17.5	15	17.2	1.9	15.3
YBEML-T1	1943	6	19.2	20	19.3	4.6	14.7
YBEML-T2	1943	5	11.6	13	11.9	1.3	10.6
YBEML-YA	1973	8	9.4	22	8.4	2.2	6.2
YETML	1997	18	28.7	53	30.5	3.6	26.9
YIEIO-A	1984	7	11.8	21	12.0	1.8	10.2
YIEIO-B	1984	7	7.9	17	10.9	2.2	8.7
YIEIO-C	1984	7	14.2	21	12.6	2.3	10.3
ZNMIO	1985	10	9.0	21	10.0	1.9	8.1

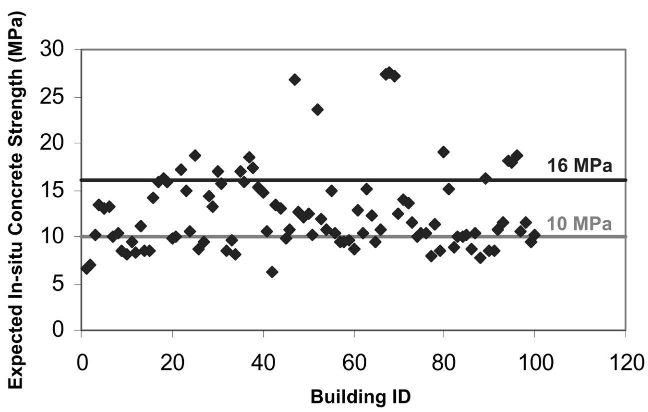

Fig. 3 Expected in-situ concrete strength distribution of the school buildings

MPa or 18 MPa. Therefore, the strength of 16 MPa has another importance, being close to the specified strength values.

Experimental study on sampled buildings indicated that the buildings constructed per pre-modern code had Grade 220 MPa reinforcement for both longitudinal and transverse reinforcement. The yield strength of both longitudinal and transverse reinforcement is taken as 220 MPa. Strain-hardening of longitudinal reinforcement has been taken into account and the ultimate strength of the reinforcement is taken as 330 MPa (TS500 2000). Although there were extreme cases where transverse reinforcement spacing was 370 mm, the observed transverse reinforcement spacing ranged between 150 and 250 mm. Hence, two spacing values are considered as 150 and 250 mm to reflect ductile and non-ductile detailing, respectively. In this study, "poor" construction quality term is used for the buildings with 10 MPa concrete strength and 250 mm transverse reinforcement spacing while "average" construction quality refers to the buildings with16 MPa concrete strength and 150 mm transverse reinforcement spacing.

2.1 Modeling approach

Member size and reinforcements in the template design were used to model the selected buildings for nonlinear analysis. No simplifications are made for the reinforcements of members; like rounding-off or grouping members with a similar reinforcement amount. All members are modeled as given in the template design.

Nonlinear static analyses have been performed using SAP2000 Nonlinear Version 8 that is a general-purpose structural analysis program (CSI 2000). Three-dimensional model of each building is created in SAP2000 to carry out nonlinear static analysis. Beam and column elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends. SAP2000 provides default and the user-defined hinge properties options to model nonlinear behavior of components. Inel and Ozmen (2006) studied possible differences on the results of pushover analysis due to default and user-defined nonlinear component properties of typical building stock in Turkey. The default hinges use the same properties without any consideration of confinement. Inel and Ozmen (2006) observed that the displacement capacities with default-hinge properties are seem to be compatible with that of well-confined case while the default hinges overestimates the displacement capacity of poorly-confined structures. Of course, there are other choices for different confinement cases. However, the user needs to spend more effort. The authors want to remind the user for possible mistakes. If the modeled building is not well confined, the default hinges provided by SAP2000 is not suitable. Thus, this study implements user-defined hinge properties. Nonlinear behavior of shear walls is modeled using FEMA-356 guidelines.

As shown in Fig. 4, five points labeled A, B, C, D, and E define force-deformation behavior of a plastic hinge (ATC-40 1996, FEMA-356 2000, CSI 2000). The values assigned to each of these points vary depending on type of element, material properties, longitudinal and transverse steel content, and axial load level on the element. Fig. 4 represents generalized force-deformation response. Five points needs to be defined for flexural response due to potential ductile behavior. However, the axial load or shear dominant behavior is brittle and no deformation capacity is available beyond point B; the member fails as it reaches its axial load or shear strength capacity. Note that number of plastic hinges to be generated for each building is in the order of 250 and 1500.

The definition of user-defined hinge properties requires moment-curvature analysis of each element. Mander model was used for unconfined and confined concrete while typical steel stress-strain model with strain hardening for steel (Mander 1984) was implemented in moment-curvature analyses. The points B and C on Fig. 4 are related to yield and ultimate curvatures. The point B is obtained from SAP2000 using approximate component initial effective stiffness values as per ATC-40 (1996), 0.5EI and 0.70EI for beams and columns, respectively. In this study, the ultimate

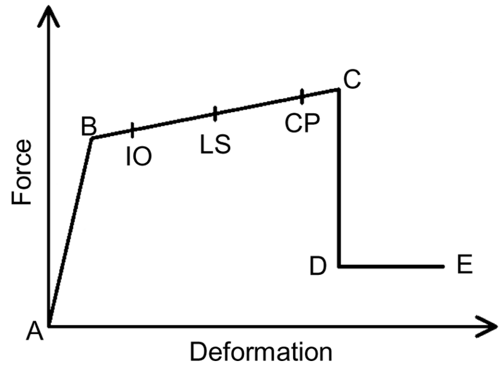

Fig. 4 Force-Deformation relationship of a typical plastic hinge

curvature is defined as the smallest of the curvatures corresponding to (1) a reduced moment equal to 80% of maximum moment, determined from the moment-curvature analysis (Priestley and Park, 1984), (2) the extreme compression fiber reaching the ultimate concrete compressive strain as determined using the simple relation provided by Priestley *et al.* (1996), given in Eqs. (2), and (3) the longitudinal steel reaching a tensile strain of 50% of ultimate strain capacity that corresponds to the monotonic fracture strain (Lehman and Moehle 2000). Ultimate concrete compressive strain (ε_{cu}) is given as

$$\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{su}}{f_{cc}} \tag{2}$$

where ε_{su} is the steel strain at maximum tensile stress, ρ_s is the volumetric ratio of confining steel, f_{yh} is the yield strength of transverse reinforcement, and f_{cc} is the peak confined concrete compressive strength.

Moment-curvature analyses were carried out considering section properties and a constant axial load on the structural element. On the beams, axial forces were assumed to be zero and on the columns they were assumed to be equal to the load due to dead load plus reduced (60 percent) live load (TEC-2007). The input required for SAP2000 is moment-rotation relationship instead of moment-curvature. Also, moment rotation data have been reduced to five-point input that brings some inevitable simplifications. Plastic hinge length is used to obtain ultimate rotation values from the ultimate curvatures. Assuming constant plastic curvature over plastic hinge length, ultimate rotation capacity, θ_u, is obtained by Eq. (3) for a typical cantilever column (Priestley *et al.* 1996).

$$\theta_u = \theta_y + \theta_p = 0.5\phi_y L + (\phi_u - \phi_y)L_p \tag{3}$$

In Eq. (3), θ_y and θ_p are yield and plastic rotation capacities, ϕ_y and ϕ_u are yield and ultimate curvatures, L_p is the plastic hinge length, L is the distance from the critical section of the plastic hinge to the point of contraflexure. Several plastic hinge lengths have been proposed in the literature (Priestley *et al.* 1996, Park and Paulay 1975, Fardis and Biskinis 2003). In this study plastic hinge length definition given in Eq. (4) which is proposed by Priestley *et al.* (1996) is used.

$$L_p = 0.08L + 0.022 f_{ye} d_{bl} \geq 0.044 f_{ye} d_{bl} \tag{4}$$

In Eq. (4), f_{ye} and d_{bl} are the expected yield strength and the diameter of longitudinal reinforcement, respectively.

Following the calculation of the ultimate rotation capacity of an element, acceptance criteria are defined as labeled IO, LS, and CP on Fig. 4. IO, LS, and CP stand for Immediate Occupancy, Life Safety, and Collapse Prevention, respectively. This study defines these three points corresponding to 10%, 60%, and 90% use of plastic hinge deformation capacity based on SEAOC Blue Book (1999) and judgment.

In existing reinforced concrete buildings, especially with low concrete strength and insufficient amount of transverse steel, shear failures of members should be taken into consideration. For this purpose, shear hinges were introduced for beams and columns. Because of brittle failure of concrete in shear, no ductility was considered for this type of hinges. Shear hinge properties were defined such that when the shear force in the member reaches its shear strength, member fails immediately. No deformation capacity is available beyond point B on Fig. 4. The shear strength of each member (V_r) is calculated according to TS-500 (2000).

$$V_r = 0.182bd\sqrt{f_c}\left(1 + 0.07\frac{N}{A_C}\right) + \frac{A_{sh}f_{yh}d}{s} \tag{5}$$

In Eq. (5), b is section width, d is effective section depth, f_c is concrete compressive strength, N is compression force on section, A_c is area of section, A_{sh}, f_{yh} and s are area, yield strength and spacing of transverse reinforcement, respectively.

2.2 Ground motions

A set of strong ground motion records from different earthquakes with different magnitudes and different Peak Ground Acceleration (PGA) values were used for seismic performance evaluation of the considered buildings. This set includes 20 records from destructive earthquakes in Turkey over past two decades and provides an opportunity to examine reasons of school building damages during those earthquakes in Turkey. Table 5 lists major attributes of records considered in this study.

Records of 1992 Erzincan, 1999 Kocaeli and Duzce earthquakes are available in corrected form. However, records of 1995 Dinar, 1998 Adana-Ceyhan, 2002 Afyon-Sultandag, and 2003 Bingol earthquakes are unprocessed. Thus, a baseline correction is required in order to obtain more reliable data. Linear base line correction and 4th order Butterworth bandpass filtering of raw acceleration records using frequencies of 0.1 and 25 Hz are processed by SeismoSignal software (Antoniou and Pinho 2006). Average response spectrum of 20 ground motion records for 5% damping is plotted in Fig. 5 as well as demand spectrum provided in Turkish Earthquake Code-2007 for design earthquake with 10% probability of exceedance in 50 years. As seen in the figure, average spectrum for the considered records is lower than the code spectrum of design earthquake (approximately 80-85%) within the period of interest for the school buildings. The code spectrum is provided to visualize the demand of selected records. No special effort has been given to fit the average of selected records to the code spectrum.

2.3 Pushover analysis

The pushover analysis consists of the application of gravity loads and a representative lateral load pattern. Gravity loads were in place during lateral loading. In all cases, lateral forces were applied monotonically in a step-by-step nonlinear static analysis. The applied lateral forces were proportional to the product of mass and the first mode shape amplitude at each story level under

Table 5 Major ground motion records from destructive earthquakes in Turkey over past two decades

Identifier	Earthquake	Date (dd/mm/yy)	Magnitude	Station	Component (°)	PGA (g)	PGV (m/s)	Dist. (km)
AF02SULT.360	Afyon-Sultandag	03.02.2002	$M_W = 6.5$	Afyon	North	0.114	0.110	73.9[1]
AF02SULT.090	Afyon-Sultandag	03.02.2002	$M_W = 6.5$	Afyon	East	0.094	0.086	73.9[1]
BN03BING.360	Bingol	01.05.2003	$M_W = 6.4$	Bingol	North	0.546	0.449	10.5[1]
BN03BING.090	Bingol	01.05.2003	$M_W = 6.4$	Bingol	East	0.277	0.199	10.5[1]
AD98CEYH.090	Adana-Ceyhan	27.06.1998	$M_S = 5.9$	Ceyhan	East	0.274	0.200	32.0[1]
AD98CEYH.180	Adana-Ceyhan	27.06.1998	$M_S = 5.9$	Ceyhan	South	0.223	0.250	32.0[1]
DN95DINA.090	Dinar	01.10.1995	$M_S = 5.9$	Dinar	East	0.330	0.360	10.8[1]
DN95DINA.180	Dinar	01.10.1995	$M_S = 5.9$	Dinar	South	0.282	0.276	10.8[1]
DZ99BOLU.360	Duzce	12.11.1999	$M_W = 7.2$	Bolu	360°	0.728	0.564	17.6[2]
DZ99BOLU.090	Duzce	12.11.1999	$M_W = 7.2$	Bolu	090°	0.822	0.621	17.6[2]
DZ99DUZC.180	Duzce	12.11.1999	$M_W = 7.2$	Duzce	180°	0.348	0.600	8.2[2]
DZ99DUZC.270	Duzce	12.11.1999	$M_W = 7.2$	Duzce	270°	0.535	0.835	8.2[2]
ER92ERZN.360	Erzincan	13.03.1992	$M_S = 6.8$	Erzincan	North	0.515	0.840	2.0[2]
ER92ERZN.090	Erzincan	13.03.1992	$M_S = 6.8$	Erzincan	East	0.496	0.643	2.0[2]
KC99DUZC.180	Kocaeli	17.08.1999	$M_S = 7.4$	Duzce	180°	0.312	0.589	12.7[2]
KC99DUZC.270	Kocaeli	17.08.1999	$M_S = 7.4$	Duzce	270°	0.358	0.464	12.7[2]
KC99GEBZ.180	Kocaeli	17.08.1999	$M_S = 7.4$	Gebze	180°	0.244	0.503	17.0[2]
KC99IZMT.090	Kocaeli	17.08.1999	$M_S = 7.4$	Izmit	090°	0.220	0.298	4.8[2]
KC99YARM.060	Kocaeli	17.08.1999	$M_S = 7.4$	Yarimca	060°	0.268	0.657	2.6[2]
KC99YARM.330	Kocaeli	17.08.1999	$M_S = 7.4$	Yarimca	330°	0.349	0.622	2.6[2]

[1]Distance to epicenter
[2]Closest distance to fault rupture

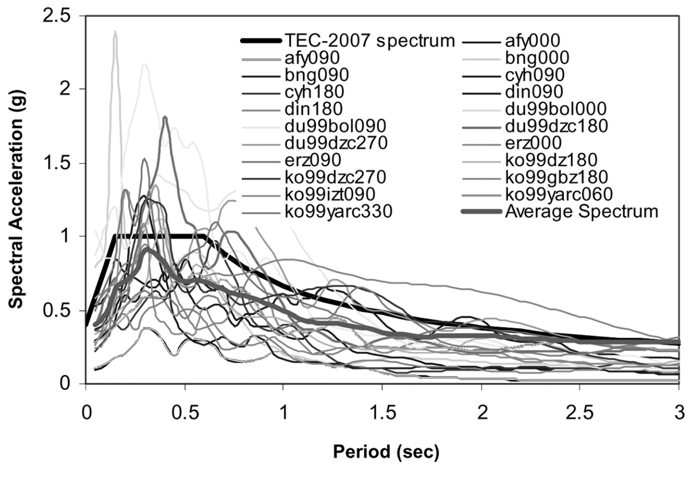

Fig. 5 Average response spectra of 20 records for 5% damping

consideration. P-Delta effects were taken into account.

In pushover analysis, the behavior of structure is characterized by a capacity curve that represents the relationship between the base shear force and the displacement of the roof. This is a very convenient representation in practice and can be easily visualized by the engineer. It is recognized that the roof displacement was used for the capacity curve because it is widely accepted in practice. For capacity curve plots, the vertical axis plots shear strength coefficient that is the base shear normalized by building seismic weight while the horizontal axis plots global displacement drift that is lateral displacement of building at the roof level normalized by building height. Capacity curve of each building considered in this study was obtained for different concrete strength and transverse reinforcement spacing mentioned in material properties section; three concrete strength and two transverse reinforcement spacing values were taken into account. The analyses of six buildings in two principal direction resulted in 72 capacity curves. The notation in figures and tables corresponds to concrete strength in MPa and transverse reinforcement spacing in mm. For example, the C10-s150 means that the building with 10 MPa concrete strength (C10) and 150 mm transverse reinforcement spacing (s150). This section gives a brief summary while Inel *et al.* (2008a) provides detailed capacity evaluation of the investigated buildings.

Two extreme cases were considered to better understand the boundaries of behavior for typical

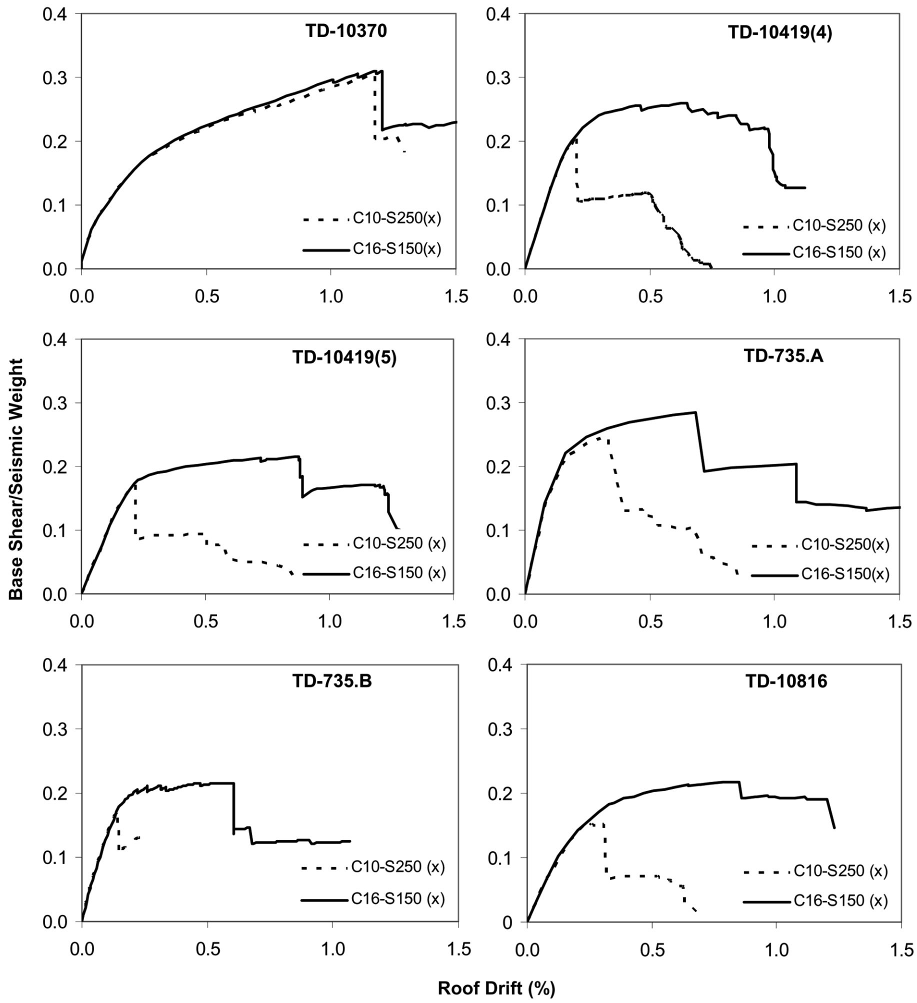

Fig. 6 Capacity curves of the buildings in poor and average states in longitudinal direction (x-direction)

school buildings with the considered template designs; poor (C10-s250) and (C16-s150) average construction quality. Capacity curves corresponding to poor and average conditions are illustrated in Figs. 6 and 7 for both longitudinal and transverse directions. The effect of transverse reinforcement spacing on displacement capacity is obvious in direction with no or relatively small shear wall amount as seen in Figs. 6 and 7. Considerably small displacement capacity for 250 mm transverse reinforcement spacing in TD-10419(4), TD-10419(5) and TD-735.B is attributed to shear failure of the columns based on the pushover analysis. Since the amount of transverse reinforcement is not enough to prevent shear failure and to provide ductile flexural response, such brittle behavior occurs. Although it is difficult to suggest a numerical limit for the amount of shear walls, the observations from Figs. 6 and 7 indicate that shear walls dominate the building response in buildings with shear wall area of at least 0.25% of total building area.

In direction with no shear walls or relatively small area of shear walls (less than 0.25% of total building area), evaluation of the capacity curves for the investigated buildings points out that: (1) Concrete quality and detailing has significant role in both displacement and lateral strength capacity of

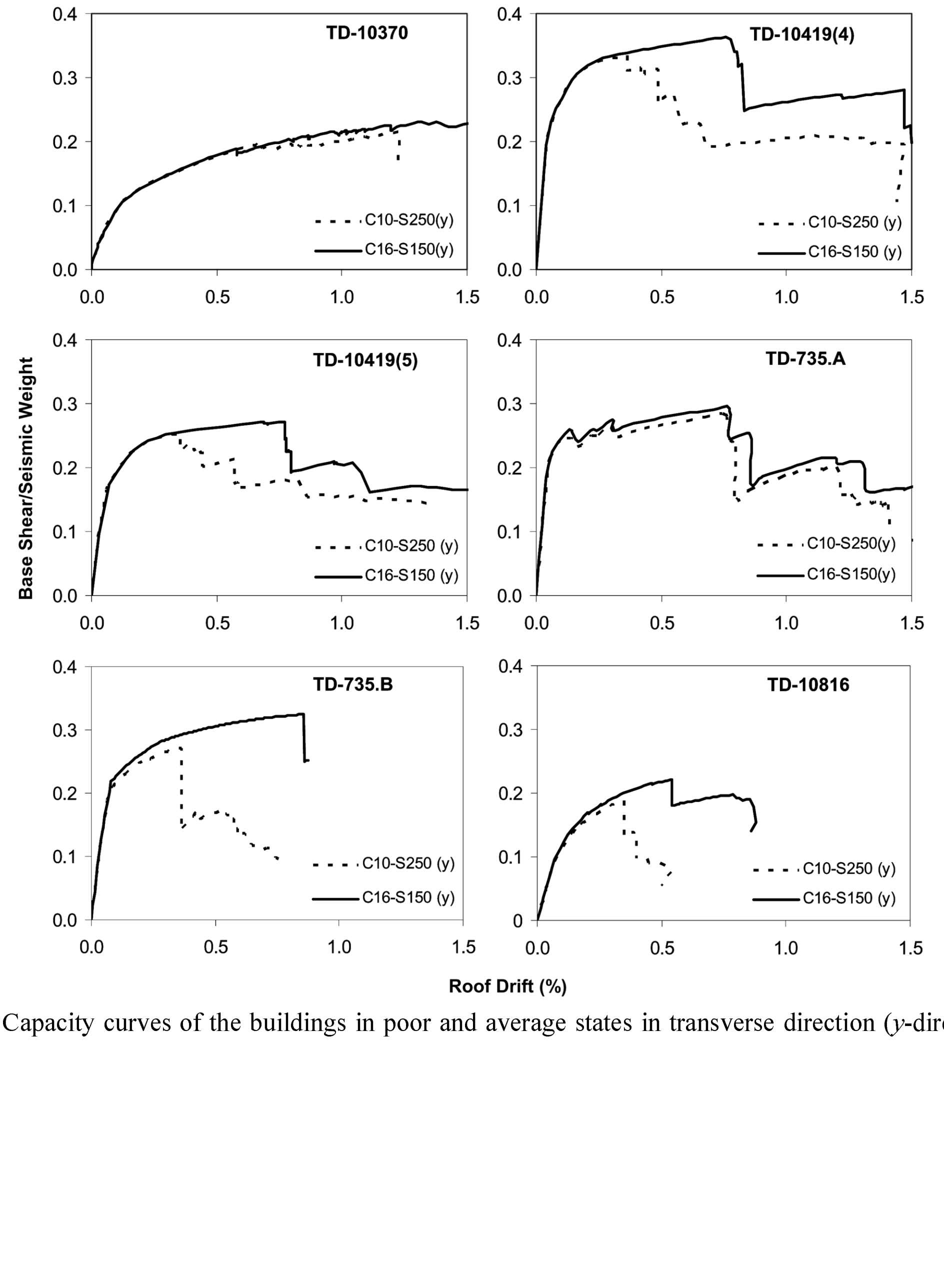

Fig. 7 Capacity curves of the buildings in poor and average states in transverse direction (y-direction)

buildings. When the worst and the best cases are examined Figs. 6 and 7, differences up to 25% on the lateral strength and more than 300% on the displacement capacity can be seen depending on concrete and construction quality. (2) Although the difference of poor and average construction quality cases on lateral strength capacity is limited, the difference in displacement capacity is noteworthy. The displacement capacity for average condition is more than twice of that for poor condition.

In direction with significant amount of shear walls (at least 0.25% of total building area), the capacity curves suggest that the concrete strength and detailing have limited effect on both lateral strength and displacement capacity. In this study, displacement capacity of some buildings with significant amount of shear wall areas seem to be somewhat below than a practitioner may expect. There are two major reasons for that. The first one is that the shear walls in these buildings are designed and detailed per pre-modern building codes and have no confined boundary elements as suggested in the modern code. Lack of confined boundary elements can decrease the plastic rotation capacity of shear walls more than 50% (FEMA-356 2000). The low concrete strength and small amount of transverse reinforcement assumed in some cases results in very brittle behavior of shear walls that controls building behavior. The second reason is that school buildings have higher dead and live loads compared to residential buildings with same amount of shear walls, resulting in inferior seismic capacity.

Buildings with shear walls are expected to have higher base shear capacity and lower displacement capacity than the bare frame buildings. In this study the buildings or directions with shear walls may have higher displacement capacities. This is related to the failure mechanism of the bare frame buildings. As the pre-modern codes per which the investigated buildings are designed did not enforce strong column-weak beam behavior, no effort has been given in detailing to prevent yielding of columns before beams. This may cause non-ductile behavior of frame systems when compared to the systems with shear walls.

2.4 Performance evaluation

Evaluation of the investigated school buildings is performed using recently published Turkish Earthquake Code (2007). Three performance levels, Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) are considered as specified in this code and several other international guidelines (ATC-40 1996, FEMA-356 2000, FEMA-440 2005). Criteria given in the code for three performance levels are listed in Table 6.

Pushover analysis data and criteria of Table 6 were used to determine global displacement drift ratio (defined as lateral displacement at roof level divided by building height) of each building corresponding to the performance levels considered. Table 7 lists global displacement capacity of each building. The capacity curve of each building is approximated with a bilinear curve using engineering judgment and equal area criterion under the actual and bilinear curves as recommended in ATC-40 (1996) and FEMA-356 (2000) guidelines. The yielding on the capacity curve is described as the point where the building starts to soften. A typical example of pushover and idealized capacity curves is shown in Fig. 8. Yield and ultimate response points represent the idealized capacity curve.

2.5 "Equivalent" SDOF idealization of building response

Yield strength coefficient, yield displacement and post-yield stiffness parameters describe

Table 6 Performance levels and criteria provided in Turkish Earthquake Code-2007 (TEC-2007)

Performance Level	Performance Criteria
Immediate Occupancy (IO)	1. There shall not be any beams beyond LS. 2. There shall not be any column or shear walls beyond IO level. 3. The ratio of beams in IO-LS region shall not exceed 10% in any story.
Life Safety (LS)	1. The ratio of beams in LS-CP region shall not exceed 20% in any story. 2. In any story, the shear carried by columns or shear walls in LS-CP region shall not exceed 20% of story shear. This ratio can be taken as 40% for roof story. 3. In any story, the shear carried by columns or shear walls yielded at both ends shall not exceed 30% of story shear. 4. There shall not be any columns or shear walls beyond CP.
Collapse Prevention (CP)	1. The ratio of beams beyond CP region shall not exceed 20% in any story. 2. In any story, the shear carried by columns or shear walls beyond CP region shall not exceed 20% of story shear. This ratio can be taken as 40% for roof story. 3. In any story, the shear carried by columns or shear walls yielded at both ends shall not exceed 30% of story shear.

"equivalent" SDOF models of buildings. FEMA-356 and ATC-40 provide guidance for "equivalent" SDOF representation of building capacity curve. While yield displacement representation of "equivalent" SDOF system is the same for both FEMA-356 and ATC-40 documents, yield strength coefficient representations differ. FEMA-440 (2005) compared performance of both "equivalent" SDOF systems and recommends the use of ATC-40 representation. Thus, the capacity curve of each building generated for the first mode vector was converted to an "equivalent" SDOF system using ATC-40 representation in which yield displacement, Δ_y, and yield strength coefficients, C_y, are given by

$$\Delta_y = \frac{\Delta_{y,roof}}{\Gamma_1} \tag{6}$$

$$C_y = \frac{S_a}{g} = \frac{V_{y,mdof}/W}{\alpha_1} \tag{7}$$

where $\Delta_{y,roof}$ = the roof displacement at yield, Γ_1 = the first (predominant) mode participation factor, S_a = the pseudo-acceleration associated with yield of the "equivalent" SDOF system, g = the acceleration of gravity, $V_{y,mdof}$ = the base shear strength of the multi-degree-of-freedom (MDOF) system or building at global yield, W = seismic weight of the MDOF system, and α_1 = the modal mass coefficient of the first mode.

2.6 Nonlinear dynamic response history analyses and performance evaluation

The inelastic dynamic characteristic of each building is represented by a bilinear "equivalent" SDOF system. The seismic displacement demand is obtained subjecting such SDOF system to time history analysis under selected ground motions listed in Table 5 without any scaling. A total of 1440 "equivalent" SDOF nonlinear response history analyses were carried out using USEE-2001 (Inel *et al.* 2001). The "equivalent" SDOF displacement demands were then converted into building displacement demands at the roof level multiplying by the first mode participation factor, Γ_1.

Following displacement demand estimates, seismic performance of each school building were

Table 7 Global displacement drift capacities (%) of the investigated school buildings obtained from capacity curves for considered performance levels

Template Design ID	Material Quality	X-direction			Y-direction		
		IO	LS	CP	IO	LS	CP
		$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$
TD-10370	C10-S150	0.15	0.56	1.16	0.20	0.44	1.40
	C10-S250	0.14	0.52	1.11	0.13	0.43	1.16
	C13-S150	0.15	0.56	1.16	0.20	0.45	1.40
	C13-S250	0.14	0.52	1.11	0.14	0.44	1.31
	C16-S150	0.31	0.61	1.16	0.20	0.46	1.77
	C16-S250	0.16	0.57	1.11	0.17	0.45	1.74
TD-10419(4)	C10-S150	0.16	0.40	0.70	0.16	0.47	0.51
	C10-S250	0.16	0.19	0.22	0.14	0.36	0.39
	C13-S150	0.19	0.49	0.73	0.18	0.47	0.70
	C13-S250	0.17	0.23	0.26	0.18	0.37	0.62
	C16-S150	0.19	0.52	0.90	0.23	0.55	0.82
	C16-S250	0.17	0.25	0.27	0.21	0.51	0.82
TD-10419(5)	C10-S150	0.29	0.39	0.43	0.28	0.43	0.52
	C10-S250	0.21	0.23	0.23	0.24	0.38	0.38
	C13-S150	0.29	0.55	0.59	0.28	0.41	0.70
	C13-S250	0.24	0.25	0.26	0.25	0.41	0.61
	C16-S150	0.31	0.73	0.94	0.31	0.45	0.82
	C16-S250	0.29	0.38	0.40	0.28	0.43	0.81
TD-735.A	C10-S150	0.16	0.27	0.41	0.13	0.25	0.75
	C10-S250	0.16	0.25	0.33	0.13	0.22	0.75
	C13-S150	0.25	0.33	0.61	0.13	0.25	0.80
	C13-S250	0.16	0.33	0.49	0.13	0.22	0.80
	C16-S150	0.25	0.50	0.69	0.17	0.30	0.80
	C16-S250	0.25	0.41	0.69	0.13	0.25	0.80
TD-735.B	C10-S150	0.15	0.32	0.38	0.16	0.41	0.74
	C10-S250	0.11	0.11	0.14	0.16	0.27	0.36
	C13-S150	0.19	0.40	0.57	0.20	0.46	0.84
	C13-S250	0.12	0.12	0.15	0.18	0.38	0.76
	C16-S150	0.22	0.42	0.60	0.21	0.50	0.86
	C16-S250	0.12	0.13	0.16	0.20	0.45	0.84
TD-10816	C10-S150	0.21	0.38	0.78	0.13	0.34	0.41
	C10-S250	0.17	0.25	0.31	0.13	0.17	0.38
	C13-S150	0.22	0.43	1.04	0.13	0.35	0.44
	C13-S250	0.18	0.27	0.35	0.13	0.20	0.39
	C16-S150	0.22	0.43	1.14	0.13	0.36	0.52
	C16-S250	0.21	0.30	0.35	0.13	0.20	0.47

evaluated using Table 7 for ground motions considered in this study. Statistics of performance evaluation is given in Tables 8 and 9 for LS and CP performance levels. In the tables, the term “exceedance ratio” refers to the number of building cases which do not comply with a given

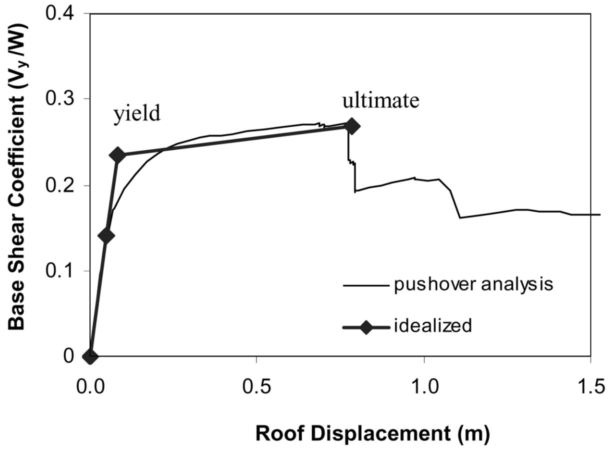

Fig. 8 Typical pushover and idealized capacity curves

performance level divided by the total number of buildings. Although the detailed statistics for Immediate Occupancy is not provided in the paper, this performance level is not satisfied for most of records.

3. Discussion of results

The observed school building damages during the past earthquakes in Turkey were reported in many studies (e.g. Dogangun 2004, EERI Special Earthquake Report 2003, Hassan and Sozen 1997, Ozcebe *et al.* 2004). Although a considerable number of school buildings at various damage levels was reported especially during 1992 Erzincan and 2003 Bingol earthquakes, no statistics were available related to total number of schools in the region. Due to extremely high number of casualties and damaged buildings, the school buildings were not covered separately for 1999 Kocaeli and 1999 Duzce earthquakes. The exceedance ratios for LS and CP performance levels observed in Tables 8 and 9 support high damaging property of 1992 Erzincan, 1999 Kocaeli and 1999 Duzce earthquakes for existing school buildings. On the other hand, the damage from the Bingol earthquake is not related to only its detrimental characteristic. The analytical results obtained in this study imply that the Bingol earthquake was especially destructive for the buildings with poor concrete quality and unacceptable transverse reinforcement spacing within potential plastic hinge regions. The results at LS performance level agree with shear failures due to insufficient amount of transverse reinforcement observed during Bingol earthquake while the results at CP level are consistent with heavily damaged or collapsed buildings due to poor concrete and inadequate transverse reinforcement spacing as reported in many studies such as Dogangun (2004), EERI Special Earthquake Report (2003) and Ozcebe *et al.* (2004).

Careful assessment of Table 8 supports the observed damages in the past earthquakes. Among twenty records considered herein, BN03BING.360, DN95DINA.090, DZ99DUZC.180, KC99DUZC.180, KC99DUZC.270, KC99YARM.060 and KC99YARM.330 records have significant damaging effects with exceedance ratio of LS performance level greater than about 0.60 in most cases while DZ99BOLU.360, DZ99BOLU.090, DZ99DUZC.270, ER92ERZN.360 and ER92ERZN.090 records are extremely destructive with exceedance ratio of LS performance level greater than about 0.80.

Table 8 Exceedance ratio of Life Safety performance levels for the investigated school buildings subjected to the selected ground motions

Identifier	Life Safety					
	C10		C13		C16	
	s = 250 mm	s = 150 mm	s = 250 mm	s = 150 mm	s = 250 mm	s = 150 mm
AF02SULT.360	0.25	0.00	0.00	0.00	0.00	0.00
AF02SULT.090	0.25	0.00	0.00	0.00	0.00	0.00
BN03BING.360	0.75	0.58	0.67	0.33	0.50	0.25
BN03BING.090	0.42	0.00	0.25	0.00	0.08	0.00
AD98CEYH.090	0.58	0.25	0.58	0.25	0.42	0.08
AD98CEYH.180	0.42	0.33	0.42	0.25	0.42	0.17
DN95DINA.090	0.67	0.58	0.67	0.42	0.50	0.33
DN95DINA.180	0.50	0.25	0.42	0.00	0.42	0.00
DZ99BOLU.360	0.92	0.83	0.92	0.83	0.75	0.67
DZ99BOLU.090	0.92	0.92	0.92	0.92	0.92	0.92
DZ99DUZC.180	0.67	0.67	0.67	0.67	0.58	0.50
DZ99DUZC.270	1.00	0.92	1.00	0.83	0.83	0.83
ER92ERZN.360	0.92	0.92	0.92	0.92	0.92	0.92
ER92ERZN.090	0.92	0.92	0.92	0.83	0.92	0.83
KC99DUZC.180	0.58	0.58	0.58	0.50	0.50	0.17
KC99DUZC.270	0.75	0.58	0.67	0.50	0.58	0.33
KC99GEBZ.180	0.33	0.00	0.25	0.00	0.25	0.00
KC99IZMT.090	0.42	0.00	0.42	0.00	0.33	0.00
KC99YARM.060	0.58	0.58	0.50	0.25	0.50	0.25
KC99YARM.330	0.67	0.42	0.58	0.25	0.50	0.17

Similar observations are valid for CP level (Table 9) with smaller exceedance ratios.

According to Turkish Earthquake Code (2007), school buildings are expected to satisfy IO and LS performance levels under design and extreme earthquakes, corresponding to 10% and 2% probability of exceedance in 50 years, respectively. Average exceedance ratio of considered performance levels is summarized in Table 10 for global evaluation of material properties. Although the records are not classified in terms of earthquake level, the average spectrum of the considered records is lower than the spectrum of design earthquake corresponding to 10% probability of exceedance in 50 years specified in TEC-2007 (Fig. 5). As seen in Table 10, IO performance level is exceeded in majority of buildings. The average exceedance ratio ranges from 0.68 to 0.83, corresponding to average (C16s150) and poor (C10s250) construction quality cases, respectively. LS performance level is exceeded in most of the school buildings even though the selected records have much lower average spectrum than TEC-2007 requires for the extreme earthquake that is one and half times design earthquake spectrum. The average exceedance ratio for LS performance level ranges from 0.32 to 0.63.

Although Collapse Prevention (CP) performance level is not desired for the school buildings, it is an important criterion for limiting casualties during an earthquake. The exceedance ratio for CP performance level ranges between 0.13 and 0.45. As material quality gets better, performance of buildings improves (Table 10). Also Table 10 evidently indicates that concrete quality and transverse reinforcement spacing have limited effect on IO level while amount of transverse reinforcement

Table 9 Exceedance ratio of Collapse Prevention performance levels for the investigated school buildings subjected to the selected ground motions

Identifier	Collapse Prevention					
	C10		C13		C16	
	s = 250 mm	s = 150 mm	s = 250 mm	s = 150 mm	s = 250 mm	s = 150 mm
AF02SULT.360	0.00	0.00	0.00	0.00	0.00	0.00
AF02SULT.090	0.00	0.00	0.00	0.00	0.00	0.00
BN03BING.360	0.42	0.17	0.42	0.17	0.42	0.00
BN03BING.090	0.17	0.00	0.17	0.00	0.08	0.00
AD98CEYH.090	0.42	0.08	0.33	0.00	0.25	0.00
AD98CEYH.180	0.42	0.08	0.42	0.08	0.33	0.00
DN95DINA.090	0.50	0.17	0.42	0.08	0.42	0.08
DN95DINA.180	0.33	0.08	0.33	0.00	0.33	0.00
DZ99BOLU.360	0.67	0.50	0.42	0.33	0.42	0.17
DZ99BOLU.090	0.83	0.75	0.75	0.75	0.75	0.75
DZ99DUZC.180	0.50	0.42	0.42	0.42	0.42	0.17
DZ99DUZC.270	0.83	0.67	0.75	0.50	0.58	0.33
ER92ERZN.360	0.83	0.75	0.75	0.67	0.58	0.58
ER92ERZN.090	0.75	0.50	0.50	0.42	0.42	0.33
KC99DUZC.180	0.42	0.17	0.33	0.08	0.33	0.00
KC99DUZC.270	0.58	0.25	0.42	0.08	0.42	0.08
KC99GEBZ.180	0.17	0.00	0.17	0.00	0.17	0.00
KC99IZMT.090	0.33	0.00	0.25	0.00	0.17	0.00
KC99YARM.060	0.42	0.17	0.42	0.00	0.33	0.00
KC99YARM.330	0.33	0.08	0.33	0.08	0.33	0.00

Table 10 Average exceedance ratio of considered performance levels for different material properties

Material Quality	Immediate Occupancy	Life Safety	Collapse Prevention
C10-S250	0.83	0.63	0.45
C13-S250	0.79	0.57	0.38
C16-S250	0.75	0.50	0.34
C10-S150	0.79	0.47	0.24
C13-S150	0.74	0.39	0.18
C16-S150	0.68	0.32	0.13

plays an important role in seismic performance of buildings for LS and CP levels as expected. The average exceedance ratios double when 250 mm spacing is used instead of 150 mm for all concrete strength values for CP level. Although closer spacing considered in this study ($s = 150$ mm) do not fully comply with the code requirements, it considerably improves building performance (Table 10). This observation implies that structures with transverse reinforcement according to modern code requirements definitely have a better seismic performance. Table 10 also shows that the existing school buildings are far from satisfying IO performance level during a possible earthquake having similar characteristic with past events. Furthermore, at least half of existing school buildings is critical for LS level suggesting that urgent planning and response need to be in action.

Average exceedance ratio of considered performance levels is listed in Table 11 according to

Table 11 Average exceedance ratio of considered performance levels for poor and average construction quality cases of different template designs

Template Design ID	Direction	Shear wall area (% of total floor area)	Immediate occupancy		Life safety		Collapse prevention	
			Poor	Avg.	Poor	Avg.	Poor	Avg.
TD-10370	Longitudinal (x)	0.37	1.00	0.70	0.35	0.20	0.15	0.10
	Transverse (y)	0.26	1.00	0.90	0.65	0.60	0.20	0.05
TD-10419(4)	Longitudinal (x)	---	1.00	1.00	1.00	0.45	0.85	0.20
	Transverse (y)	0.38	0.50	0.25	0.10	0.00	0.05	0.00
TD-10419(5)	Longitudinal (x)	---	0.90	0.75	0.85	0.30	0.85	0.15
	Transverse (y)	0.30	0.45	0.20	0.20	0.20	0.20	0.05
TD-735.A	Longitudinal (x)	0.23	0.90	0.70	0.70	0.20	0.50	0.15
	Transverse (y)	0.37	0.55	0.50	0.45	0.25	0.00	0.00
TD-735.B	Longitudinal (x)	0.13	1.00	0.80	1.00	0.35	0.90	0.25
	Transverse (y)	0.25	0.65	0.50	0.30	0.15	0.30	0.00
TD-10816	Longitudinal (x)	0.10	1.00	0.90	0.90	0.60	0.80	0.15
	Transverse (y)	0.20	1.00	1.00	1.00	0.55	0.55	0.40

template designs for further performance evaluation. The numbers in the table include poor (C10s250) and average (C16s150) construction quality cases. In performance evaluation for average exceedance ratio, the performance level is considered to be satisfied if the average exceedance ratio is lower than 0.50. Except transverse direction of TD-10419(4) and TD-10419(5), IO level is not satisfied for the considered template designs. TD-10816 (both directions) and longitudinal direction of TD-10419(4), TD-10419(5), TD-735.A and TD-735.B are definitely critical for LS and CP performance levels when the poor construction quality case is considered. Transverse direction of TD-10370 does not assure LS performance level in both cases. The exceedance ratios given in the table clearly shows that performance of buildings improves as material quality gets better. Except both directions of TD-10816 and the transverse direction of and TD-10370, all templates with average construction quality assure LS performance levels. Besides, CP is satisfied in all templates with the average construction quality.

The capacity curves of template designs are revisited to identify possible deficiencies and their solutions. Each pushover curve is carefully examined at LS and CP performance levels. It has been seen that small enhancements to increase ductility of several columns will improve the performance of TD-10370 in transverse direction. However, longitudinal direction of TD-10419(4) and TD-10419(5) has considerably small displacement capacity, especially for 250 mm transverse reinforcement spacing. Shear failures in columns are observed. As aforementioned, these template designs have no shear walls in this direction. Additional shear walls definitely take earthquake effects and reduce the burden of columns. Moreover, critical columns need to be enhanced for shear failures. Although longitudinal direction of TD-735.B has shear walls (0.5% of floor area), they are not sufficient to relieve columns. Similar observations are valid for both direction of TD-10816. It should be noted that this building has shear walls as 1% of floor area. Additional shear walls may be considered for longitudinal direction of TD-735.B and both directions of TD-10816.

Average exceedance ratios were plotted against amount of shear walls normalized by total area of buildings for the considered performance levels and provided in Fig. 9. As seen in the figure, building performance improves as the amount of shear wall increases. This emphasizes its

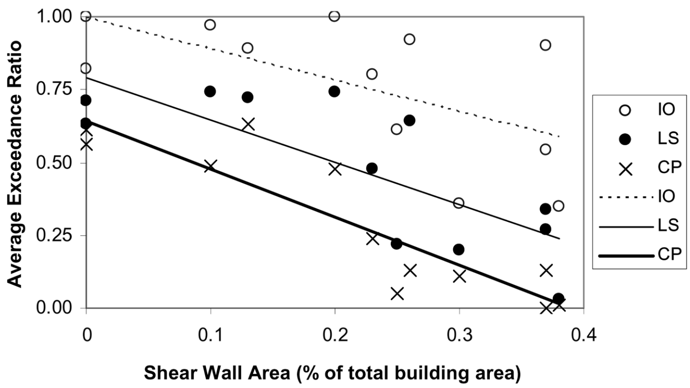

Fig. 9 Average exceedance ratio for a given performance level versus shear wall area (% of total building area). Lines on the figures are the best fit from linear regression

importance, especially in countries where construction with poor detailing is a common problem. The use of shear walls increases lateral load capacity and significantly decreases displacement demands while existing deficiencies in frame elements are less pronounced and poor construction quality in buildings is somewhat compensated.

4. Conclusions

This study evaluated seismic performance of the school buildings with the selected template designs in Turkey considering nonlinear behavior of reinforced concrete components. Six school buildings with template designs were selected to represent major percentage of school buildings in medium-size cities located in high seismic region of Turkey. Selection of template designed buildings and material properties were based on field investigation on government owned school buildings in Denizli, Turkey. Capacity curves of investigated buildings were determined by pushover analyses conducted in two principal directions. The inelastic dynamic characteristics were represented by equivalent single-degree-of-freedom (SDOF) systems and their seismic displacement demands were calculated under selected ground motions. Seismic performance evaluation was carried out in accordance with recently published Turkish Earthquake Code (2007) that has similarities with FEMA-356 guidelines. Reasons of building damages in past earthquakes are examined using the results of performance assessment of investigated buildings. The observations and findings of the current study are briefly summarized in the following. It should be noted that the conclusions related to poor concrete quality are based on concrete compressive strength of 10 MPa. The adherence between concrete and reinforcing bars is not satisfied when concrete strength is too low.

1. Evaluation of the capacity curves for the investigated buildings points out that concrete quality and detailing has significant role especially in displacement capacity in direction with no shear walls or relatively small area of shear walls; the displacement capacity for average construction quality (C16 and s150) is more than twice of that for poor condition (C10 and s250).
2. Column shear failures are common problem for poor concrete and low amount of transverse reinforcement, resulting in brittle failure for existing school buildings.
3. The analytical results support that although poor concrete quality and unacceptable transverse reinforcement spacing within potential plastic hinge regions are the main parameters for poor

performance of school buildings in past earthquakes, 1992 Erzincan and 1999 Kocaeli and 1999 Duzce earthquakes have detrimental effects on the buildings.

4. As material quality gets better, performance of buildings improves. The displacement capacities obtained for different performance levels evidently indicate that concrete quality and transverse reinforcement spacing have limited effect on IO level while amount of transverse reinforcement plays an important role in seismic performance of buildings for LS and CP levels as expected by engineer.
5. Amount of transverse reinforcement is a significant parameter in seismic performance of buildings. This study shows that as the amount of transverse reinforcement increases the sustained damage decreases; the average exceedance ratios double when 250 mm spacing is used instead of 150 mm for all concrete strength values.
6. Performance evaluation of the investigated school buildings indicates that existing school buildings are far from satisfying performance objectives of the recent Turkish Earthquake Code during a possible earthquake having similar characteristic with past events.
7. Performance of school buildings improves as the amount of shear wall increases, emphasizing its importance, especially in countries where construction with poor detailing is a common problem. The use of shear walls increases lateral load capacity and significantly decreases displacement demands while existing deficiencies in frame elements are less pronounced and poor construction quality in buildings is somewhat compensated.
8. The main deficiency of existing school buildings is high displacement demands due to their low lateral load capacities, especially when there is no or limited amount of shear walls. Almost all members are therefore inadequate for displacement demand concern. Under these circumstances the addition of shear walls seems to be most practical and economical solution according to the authors' experience although there may be other solutions.

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