Seismic Analysis

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Seismic Assessment of Existing R.C. Public Buildings in Turkey – An Overview with a Case Study

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Summary

The recent devastating earthquakes have exposed the vulnerability of the existing public buildings in Turkey. A great part of these reinforced concrete buildings has been designed considering earlier codes when seismic loads were not required or the design was at lower level of seismic loads of what is currently specified. 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, hospitals, schools, etc.) as prevalent practice to save on architectural fees and ensure quality control. The need for evaluating the seismic adequacy of these public buildings has come into focus following the enormous loss of life and property during the recent earthquakes.

This paper aims to evaluate the seismic performance of a public building with the selected template design in Turkey considering the nonlinear behavior of reinforced concrete members. For the building addressed in this paper, material properties are based on field investigation on government public buildings in western part of Turkey. Seismic performance evaluation will be carried out in accordance with the recently published Turkish Earthquake Code-2007 that has many similarities with FEMA 356 guidelines.

Capacity curves of investigated building will be determined by nonlinear static analysis. The effects of material quality on seismic performance of this public building will be investigated. In conclusion, different possible deficiencies and solutions to improve template design building will be discussed. This study gives an in depth sight into to the rehabilitation of public buildings in Turkey.

KEYWORDS: Nonlinear static analysis; Public building; Reinforced concrete; Seismic Code; Seismic performance evaluation.

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1. INTRODUCTION

Considerable losses of life and properties have taken place as a result of the destruction caused by the earthquakes that have happened in Turkey during the past two decades. A large number of existing buildings in Turkey and other developing countries, built according to design codes of the 70s, shows that many of them behave poorly and have insufficient seismic safety. Particularly, damages that occurred on public buildings are more serious and irreparable compared to the damage that occurred on residential buildings. The damages that occurred to the public buildings in Erzincan earthquake of March 13, 1992; Adana-Ceyhan earthquake of June 27 1998; Marmara Earthquake of November 12, 1999 and Bingöl earthquake of May 1, 2003 [1-5] made it clear that these buildings, which are built mostly of reinforced concrete, need to be examined and retrofitted rapidly and effectively if necessary.

The projects and the construction of existing public buildings that were built before 1998 were constructed in accordance with the regulations TBC-1984 [6] and TEC-1975 [7] which were in effect at that time. In general, public buildings designed without seismic considerations have significant deficiencies, such as discontinuity of positive moment reinforcement in beams and wide spacing of transverse shear reinforcement. However, the earthquake and the construction regulations underwent significant changes with revisions made in 1998, 2000 and 2007 [8-9-10]. The strengthening of existing public buildings in conjunction with new contract specifications, thereby reducing looses of life and property to a minimum in case of an earthquake has become one of the most important issues on the agenda of Turkish Government [11]. In addition, a number of major earthquakes during last two decades in Turkey have underscored the importance of mitigation to reduce seismic risk.

Seismic retrofit of existing structures is one method to reduce the risk to vulnerable structures. Recently, a significant amount of research has been devoted to the study of various retrofit techniques to enhance the seismic performance of RC structures. However, few studies have been conducted to assess the seismic performance of representative concrete structures in Turkey using the criteria of Turkish Earthquake Code-2007 (TEC-2007) [10].

The objectives of this study are to evaluate the seismic performance of a typical 1990s RC public (hospital) building in Turkey using TEC-2007 which has many similarities with FEMA-356 [12], performance criteria and determine the various seismic retrofit techniques. Both the TEC-2007 global level and member level limits were assessed for three performance levels. In order to compute global structural parameters, such as stiffness, strength and deformation capacity; pushover analysis was conducted for the case hospital building. The results of the pushover analysis were investigated according to TEC-2007 requirements for

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evaluating the seismic response of this building. Finally, different possible retrofitting solutions able to improve the seismic behaviour of non-seismically designed public buildings have been discussed.

2. PUBLIC BUILGINGS IN TURKEY

2.1. Template Designs

In Turkey, template designs developed by the Ministry of Public Works are used in all provinces 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. For that reason, these are the buildings must be dealt with firstly. Although the used projects display minor differences from province to province, they were similar architecturally.

For example, for the school buildings a revolutionary step was taken in 1997 when the 5-year mandatory education was extended to 8 years. This transformation led to the emergence of a need for new spaces. Attempts were made to solve these problems by adapting the exiting primary schools to the 8-year ones through some physical changes or by constructing new school buildings, efforts that still continue. The most preferred method for the adaptation of existing buildings is the addition of floors.

The general and the common properties of public buildings are as follows:

- The load bearing system of these buildings is composed of reinforced concrete column and beam system.
- These buildings are constructed in accordance with TEC-1975 and TBC-1984.
- There is less or no reinforced concrete shear wall in the load bearing system to resist lateral loads and impart rigidity to the building.
- Mostly, the column members of structural frame are located on the exterior axes.

2.2. Seismic Performance of Public Buildings in Turkey

Substantial damages have occured in recent earthquakes, which led to serious doubts as to the seismic performance of public buildings [1-5]. The damages that occur in public buildings are caused by the following reasons:

- Beams stronger than columns (in terms of moment capacities),
- Compression strength of concrete is very low (7-16 MPa),
- Insufficient stirrup spacing in column and beam joints,
- Plain and improper reinforcement bars,
- The fact that the hooks of stirrups have a 90^0 in angle,

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- Insufficient column sections,
- Lack of shear walls,
- The fact that vertical load bearing elements are unidirectional.

2.3. Description of the Case Study Building

Hospitals likewise the other buildings intended for governmental services are generally constructed by applying template designs developed by the Ministry of Public Works. Therefore, a considerable number of buildings have the same template designs in different parts of Turkey.

A field survey was carried out in Sparta and Denizli to select the most common type of hospital buildings. These cities are located in a seismically active part of Turkey. According to the survey, a most common type of template design (TD-11276) for hospital buildings was selected to represent these public buildings in medium-sized cities. It is, of course, impossible to reflect all the template RC public building features of the building stock of the country with only a selected template design. However, it can be assured that the selected building should have some general properties representative of these types.

This is a four-story hospital building with a plan area of 560 square meters at the base. All floor slabs are reinforced concrete with a thickness of 0.22 m. The story heights are 3.2 m for each story. There exits no exact data about the roofing and the masonry partitions of the building. From the architectural drawing plotted at the time of construction, reasonable values are assumed for both in dead load and other calculations, considering probable changes made during the construction.

The building has a typical structural system, which consists of reinforced concrete frames with masonry infill walls of hollow clay brick units. The structural system is free of shear walls since usage of vertical elements with depth/width ratio greater than five (given by TEC-1975) is not widely preferred in construction practice concerning the overall building stock. There are no structural irregularities such as soft story, weak story, heavy overhangs, great eccentricities between mass and stiffness centers and etc. One of the possible deficiencies for this building designed per TEC-1975 [7] is the strong beam-weak column behavior as it is not regarded by that code. Fig. 1-2 provides a typical floor plan and 3-D view of this case study structure respectively.

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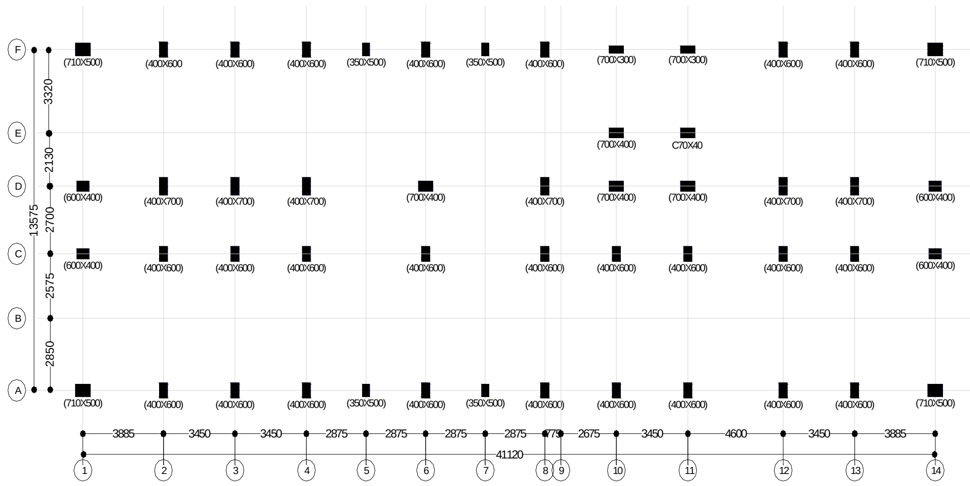

Figure 1. Typical structural floor plan view of the TD-11276 building

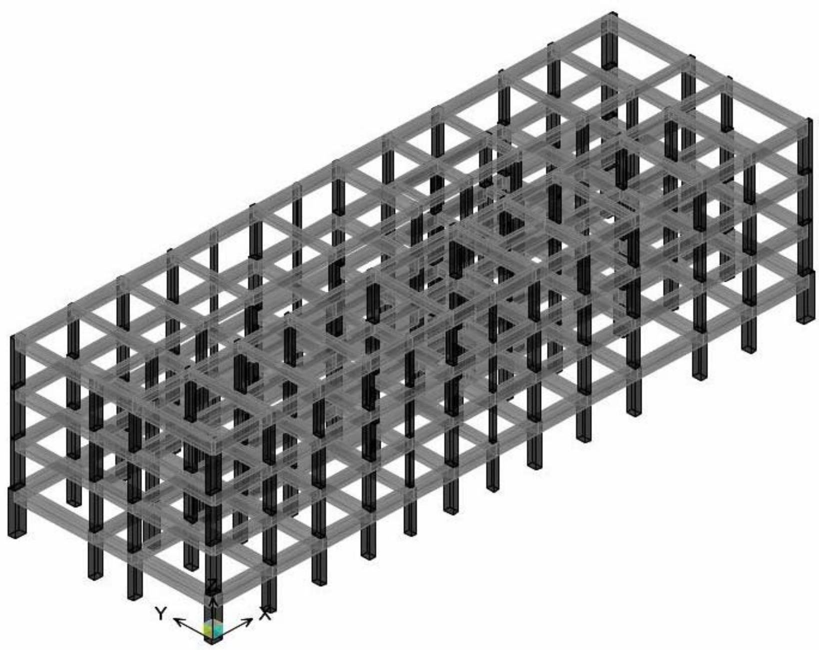

Figure 2. Three dimensional view of the TD-11276 building

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3. MODELLING OF THE CASE STUDY BUILDING

3.1. Analytical Modelling of the Case Study Building

For modelling and the analysis of the case study building, the computer program SAP2000NL [13] was employed. This is a general-purpose structural analysis program for performing static and dynamic finite element analyses of structures. In this study, Nonlinear Version 8.2.3 of the program was used. Description of the modelling details is provided below.

This building was modelled as three dimensional frame system formed by beams and columns. Frame type elements having zero mass were used for the definition of all elements in order to control total mass of the building. The representation of beam-column joints was realized by assigning rigid end offsets at the ends of the elements. The joints connecting the base columns to the foundation were restrained for all degrees of freedom assuming an infinitely rigid foundation. All joints at a floor level were constrained to move as a planar diaphragm in order to prevent in—plane membrane deformations. No slabs were defined; instead, slab weights were distrubuted to side beams as dead loads. Weights of the beams, columns, walls and the roofs were also assigned as distributed dead loads on beams. Another load case was defined to introduce live loads on beams. Masses assigned to the stories were calculated using these dead and live load values. The calculation of these masses, live loads and dead loads were made according to Turkish Standards for Reinforced Concrete, TBC-2000 [9], Turkish Standards for Design Loads, TS498 [10] and TEC-1975 [11].

For nonlinear analysis of the case building, as-built material properties determined from field investigaiton and experiment were taken into account. Material properties considered in his study were determined based on field study on 98 public buildings. Figure 3. plots the distribution of the expected concrete strength of these public buildigns. According to test results, two types of strength values, 10, 16 MPa were taken into consideration to represent typical concrete strength values for this building.

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. 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

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strength and 250 mm transverse reinforcement spacing while "average" construction quality refers to the buildings with 16 MPa concrete strength and 150 mm transverse reinforcement spacing.

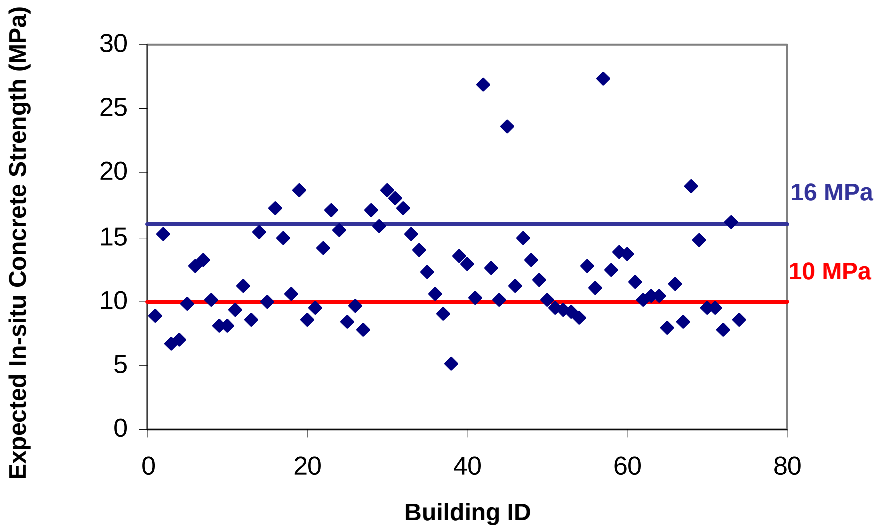

Figure 3. Expected in-situ concrete strength distribution of the school buildings

3.2. Determination of Nonlinear Parameters of Beam – Column Elements

Member size and reinforcements in the template design were used to model the sample building for nonlinear analysis. No simplifications are made for the reinforcements of members; like rounding-off or grouping members ones with close reinforcement amount. All members are modelled as given in the template design.

Three-dimensional model of the case study building is created in SAP2000 to carry out nonlinear static analysis. The structural modelling is carried out with the beam and column elements, considering the nonlinear behaviour concentrated in plastic hinges at both ends of beams and columns. SAP2000 provides default or the user-defined hinge properties options to model nonlinear behaviour of components. Inel and Ozmen [15] studied possible differences on the results of pushover analysis by implementing default and user-defined nonlinear component properties. They observed that although the model with default hinge properties seemed to provide reasonable displacement capacity for the well-confined case, the displacement capacity estimate was quite high compared to that of the poorly-confined case. Thus, this study implements user-defined hinge properties.

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The definition of user-defined hinge properties requires moment-curvarture relationships for beams and columns and axial force moment capacity data for the columns are necessary for the SAP2000 input as nonlinear properties of elements (Fig. 4).

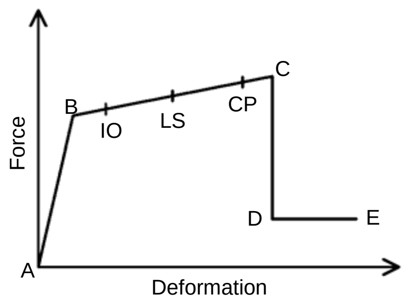

Figure 4. Typical Force Deformation Relationship

Mander model was used for unconfined and confined concrete while typical steel stress-strain model with strain hardening for steel [16] 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 [17]; 0.5EI and 0.70EI for beams and columns, respectively. In this study, the ultimate 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, (2) the extreme compression fiber reaching the ultimate concrete compressive strain as determined using the simple relation provided by Priestley et al. [18], given in Eqs. 1, and (3) the longitudinal steel reaching a tensile strain of 50% of ultimate strain capacity that corresponds to the monotonic fracture strain. Ultimate concrete compressive strain (ε_{cu}) is given as

$$\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh} \varepsilon_{su}}{f_{cc}} \qquad (1)$$

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.

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. Several plastic hinge lengths have been proposed in the literature (Priestley et al. 1996 [18]; Park and

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Paulay 1975 [19]; Fardis and Biskinis 2003 [20]). Plastic hinge length definition given in Eq. 3 which is proposed by Priestley et al. [18] is used in this study.

$$L_p = 0.08L + 0.022 f_{ye} d_{bl} \geq 0.044 f_{ye} d_{bl} \tag{2}$$

In Eq. 2, L_p is the plastic hinge length, L is the distance from the critical section of the plastic hinge to the point of contra-flexure, f_{ye} and d_{bl} are the expected yield strength and the diameter of longitudinal reinforcement.

Following the calculation of the ultimate rotation capacity of an element, acceptance criteria are represented defined as labeled IO, LS, and CP on Fig. 2. 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.

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 immediately fails. The shear strength of each member (V_r) is calculated according to TBC-2000 [9].

$$V_r = 0.182 bd \sqrt{f_c} \left(1 + 0.07 \frac{N}{A_c}\right) + \frac{A_{sh} f_{yh} d}{s} \tag{3}$$

In Eq. 3, 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.

4. EVALUATION METHODOLOGY

4.1. Nonlinear Static Procedure (Pushover Analysis)

The pushover analysis consists of the application of gravity loads and a representative lateral load pattern. The applied lateral forces were proportional to the product of mass and the first mode shape amplitude at each story level under consideration. P-Delta effects were taken into account.

In the capacity curve plots, shear strength coefficient that is the base shear normalized by building seismic weight is on the vertical axis, while global displacement drift that is lateral displacement of building at the roof level normalized by building height is on the horizontal axis. Capacity curves of the

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building considered in this study was obtained for different concrete strength and transverse reinforcement spacing mentioned in previous section; two concrete strength and two transverse reinforcement spacing values were taken into account (Fig. 5). 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).

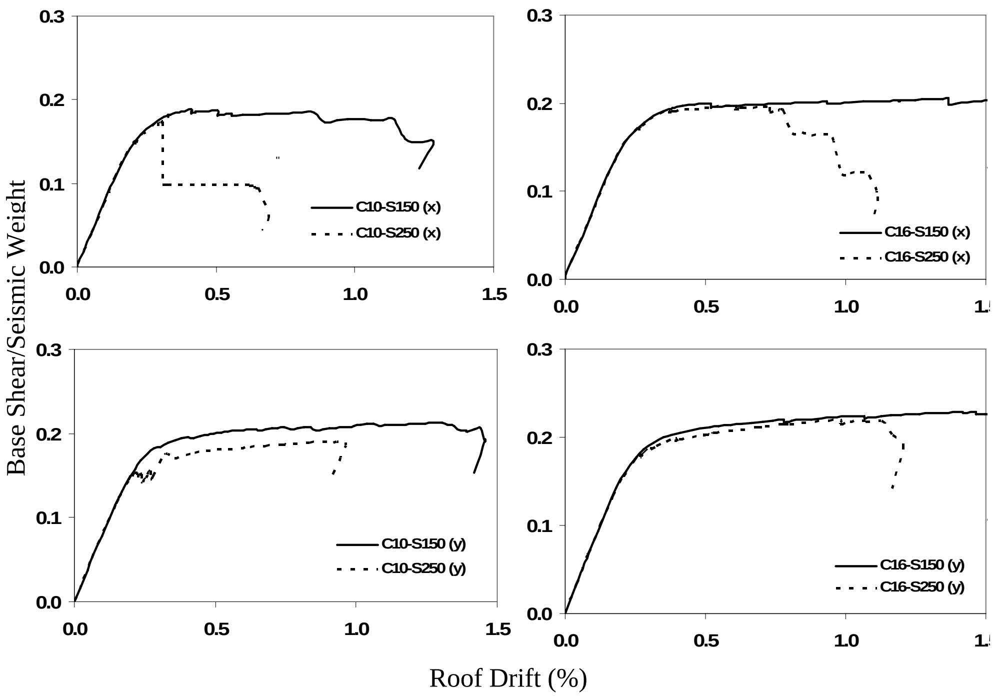

Figure 5. Capacity curves of the building TD-11276 for different concrete strength and transverse reinforcement spacing obtained by pushover analysis.

The effect of transverse reinforcement spacing on displacement capacity is obvious in longitudinal direction as seen in Fig. 5-6. Considerably small displacement capacity for 250 mm transverse reinforcement spacing is as cause of shear failure of the columns. Since the amount of transverse reinforcement is not enough to prevent shear failure and to provide ductile flexural response either, such brittle behaviour occurs. For the 150 mm spacing case, the effect of concrete strength is only limited to poor concrete case (10 MPa), having smaller displacement at significant lateral strength loss compared to the 16 MPa concrete strength.

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Two extreme cases were considered in order to have a more accurate understanding in the boundaries of behavior for the case study building with the considered template design. The first one represents the buildings in poor condition having poor concrete quality (10 MPa) with non-ductile detailing (250 mm transverse reinforcement spacing). The second one refers to the buildings in average condition having average concrete quality (16 MPa) with ductile detailing (150 mm transverse reinforcement spacing). Capacity curves corresponding to poor and average conditions are illustrated in Figs. 6 for longitudinal (x) and transverse (y) directions.

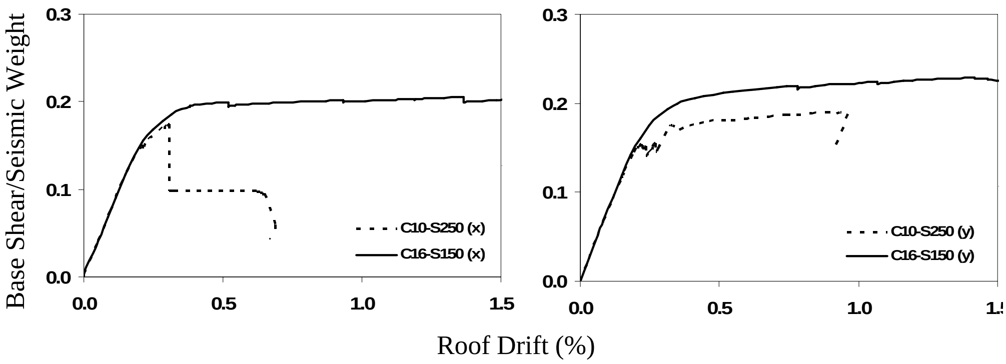

Figure 6. Capacity curves of the building TD-11276 for different concrete strength and transverse reinforcement spacing obtained by pushover analysis.

Evaluation of the capacity curves for the investigated building points out that: (1) Concrete quality and detailing has significant role in both displacement and lateral strength capacity of buildings. (2) Although the difference of poor (C10 and s250) and average (C16 and s150) conditions 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. (3) The effect of concrete strength is limited.

4.2. Capacity Assessment

Capacity assessment of the investigated case study buildings is performed using recently published TEC-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 such as FEMA-356 [12], ATC-40 [17], and FEMA-440 [21]. Criteria given in the code for three performance levels are listed in Table 1.

Table 1. Performance levels and criteria provided in Turkish Earthquake Code-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. 4. Story drift ratio shall not exceed 0.8% 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. Story drift ratio shall not exceed 2% in any story. 5. 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. 4. Story drift ratio shall not exceed 3% in any story.

Pushover analysis data and criteria of Table 1 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 2 lists global displacement drift ratios of the building. Small displacement capacities at LS and CP performance levels are remarkable for the building with poor concrete quality and less amount of transverse reinforcement due to shear failures in columns.

Table 2. Global displacement drift capacities (%) of the investigated building obtained from capacity curves for considered performance levels

Material Quality	**X-direction**			**Y-direction**		
	IO	**LS**	**CP**	**IO**	**LS**	**CP**
	$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$
C10-S150	0.27	0.33	0.84	0.26	0.28	1.32
C10-S250	0.17	0.27	0.30	0.11	0.22	0.95
C16-S150	0.34	0.54	1.51	0.28	0.47	1.44
C16-S250	0.24	0.36	0.76	0.20	0.26	1.11

The displacement capacity values are solely not meaningful themselves. They need to be compared with demand values. According to Turkish Earthquake Code, hospital 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. Response spectrum for the design and extreme earthquakes is plotted in Fig.7 for high seismicity region and soil class Z3 that is similar to class C soil of FEMA-356. Displacement demand estimates and capacities corresponding to IO and LS performance levels are compared in order to see whether the hospital building has adequate capacity.

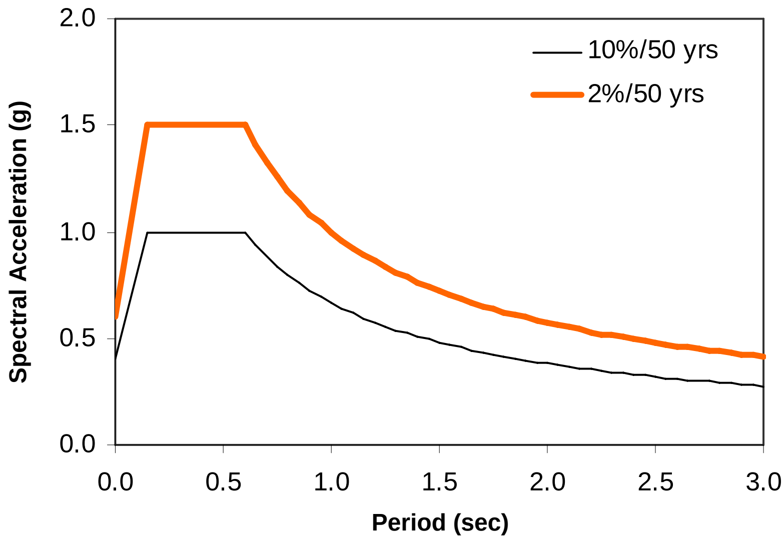

Figure 7. Response spectrum for design and extreme earthquake events provided in TEC-2007

Displacement demand estimates were obtained (Table 3) using the "equivalent" SDOF idealization of the building response as described in TEC-2007 that is similar to ATC-40.

Table 2. Global demand drift ratios (%) of the investigated building according to TEC-2007

X-direction		Y-direction	
IO	LS	IO	LS
$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$
0.93	1.40	0.93	1.40

However, the case study building constructed per TEC-1975 is far from satisfying the performance requirements of recently published code. The obvious trend between poor and average cases supports the enhancement in the building performance as the concrete quality and transverse reinforcement amount increases.

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The capacity curves of template design are revisited to identify possible deficiencies and their solutions. Each pushover curve is carefully examined at LS and CP performance levels. Longitudinal direction has considerably small displacement capacity, especially for 250 mm transverse reinforcement spacing. Shear failures in columns are observed. Additional shear walls definitely take earthquake effects and reduce the burden of columns. Moreover, critical columns need to be enhanced for shear failures.

3. CONCLUSIONS

This study evaluated seismic capacity of a typical hospital building with the selected template design constructed per pre-modern code in Turkey considering nonlinear behavior of reinforced concrete components. Selection of template designed building and material properties were based on field investigation on public buildings in several cities in western part of Turkey. Capacity curves of investigated buildings were determined by pushover analyses conducted in two principal directions. Seismic performance evaluation was carried out in accordance with recently published Turkish Earthquake Code (2007) which has similarities with FEMA-356 guidelines. Deficiencies and possible solutions to improve the capacity of this case study building are discussed. The observations and findings of the current study are briefly summarized as following;

- Evaluation of laboratory and Schmidt hammer test results obtained from 98 buildings identifies that the expected concrete strength ranges between 5.1 and 27.4 MPa while the concrete strength of most buildings is within 10 and 16 MPa ranges. Hence, two strength values, 10 and 16 MPa, were considered in this study to represent typical concrete strength values of existing hospital buildings constructed per pre-modern code.
- Field investigation on sampled buildings indicated that the buildings constructed before the modern code had Grade 220 MPa reinforcement for both longitudinal and transverse reinforcement. 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.
- Evaluation of the capacity curves for the investigated buildings points out that concrete quality and detailing has significant role in displacement and lateral strength capacity of buildings either in both directions. Although the difference of poor (C10 and s250) and average (C16 and s150) conditions on lateral strength capacity is limited, the difference in displacement capacity is

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noteworthy. The displacement capacity for average condition is more than twice of that of the poor condition.

- Shear failures of columns are common problems for poor concrete and low amount of transverse reinforcement, resulting in brittle failure for existing hospital buildings.
- The observed public building damages during the past earthquakes in Turkey support the analytical results obtained in this study; the reports from past earthquakes pointed out poor material quality and inadequate transverse reinforcement spacing within potential plastic hinge regions causing shear failures of columns. Shear failures observed in pushover analyses for the poor condition (C10s250) are clear indicators of such failures and a potential risk in existing hospitals for future earthquakes.
- According to Turkish Earthquake Code, hospital 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. The existing hospital building is far from satisfying the expected performance levels, suggesting that urgent planning and response need to be in initiated.
- 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.
- Amount of transverse reinforcement is a significant parameter in seismic performance of the buildings. This study shows that as the amount of transverse reinforcement increases the displacement capacity increases as well and therefore the sustained damage decreases.
- Adding of shear walls increases lateral load capacity and decreases displacement demands significantly. Thus, existing deficiencies in frame elements are less pronounced and poor construction quality in buildings is somehow compensated [5].

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