2nd International Balkans Conference on Challenges of Civil Engineering, BCCCE, 23-25 May 2013, Epoka University, Tirana, Albania.

Fragility Based Assessment of a School Building in Turkey

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ABSTRACT

This study focuses on, by means of a probabilistic approach, the seismic safety evaluation of a school building in Turkey designed before the introduction of modern anti-seismic codes. A typical school building which was designed according to the 1975 version of the Turkish Earthquake Code is selected for the performance assessment. Inelastic pushover and time history analyses are deployed under the effect of one hundred input ground motions. Fragility curves are generated for different concrete and detailing quality in terms of peak ground velocity. The probabilistic seismic response and vulnerability of the school are investigated by building fragility curves of the system and of its most vulnerable components. The results illustrate the significance of assessing the vulnerability of typical school buildings under the effect of various seismic scenarios and the need for extending this study to cover other typical classes of school buildings in the region.

INTRODUCTION

Recent earthquakes in Turkey have caused extensive damage on typical school buildings. Reinforced concrete (RC) school buildings constructed with template designs have been used as a common practice in Turkey [3]. The projects of existing school buildings that were built before 1998 were designed in accordance with the regulations of Turkish Earthquake Code 1975 [16]. Field observations and several studies have proved that these types of buildings are under risk [3, 4]. Thus, evaluation and reduction of the seismic vulnerability of these buildings is a task of extreme importance.

Generally, the vulnerability assessment involves the use of seismic fragility curves. These probabilistic tools provide the probability that a specified limit state or failure condition is exceeded, conditional to the strong-motion shaking severity, measured by means of an appropriately selected intensity measure. Recent studies have been developed which employ fragility curves in evaluating structural performances [1].

In this paper, the author considers a typical existing RC school building designed according to older code regulations with limited ductility. Fragility curves are built through non linear dynamic analysis for a set of different ground motion records. The findings of the study are useful for possible seismic mitigation studies in Turkey or other countries with similar construction practice and seismicity.

DESCRIPTION OF THE CASE STUDY BUILDING

A site survey was carried out in western part of Turkey to investigate the most common typical designs among school buildings. According to the survey results, the most typical design for school building was TD-10419. It was employed as 4- and 5- storey forms. In this study, 4-storey TD-10419 is used for the performance evaluation. A representative plan view of the building for the ground storey is given in Fig 1 (having shear walls only in transverse direction of the building). Typical column and wall dimension are illustrated in the figure.

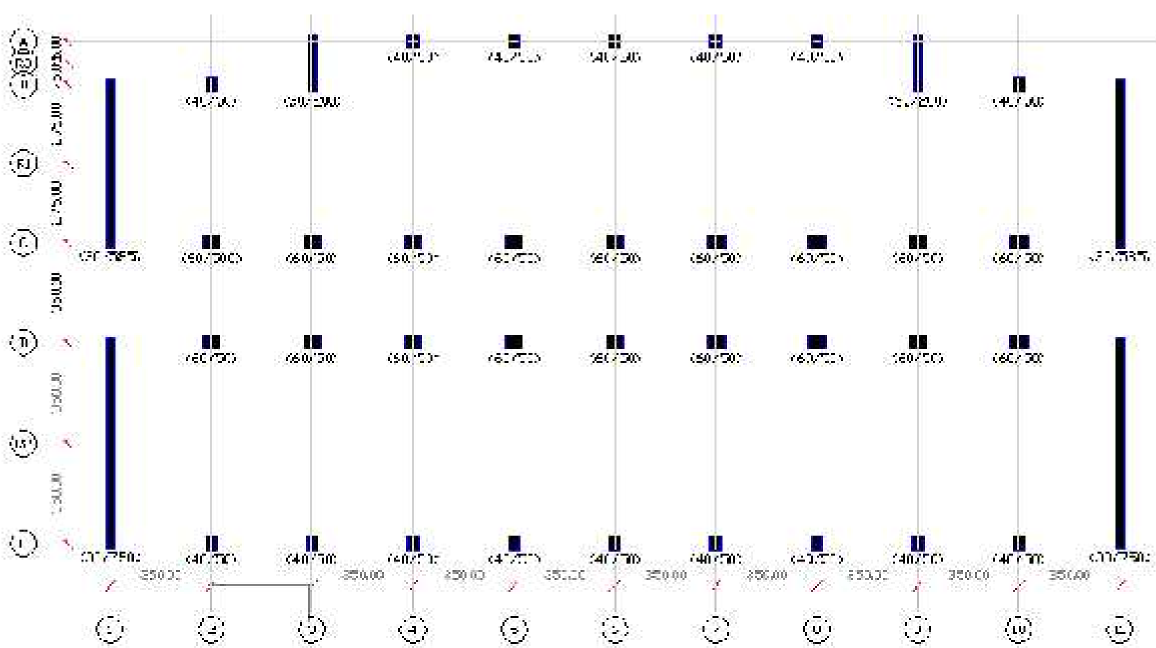


Figure 1. Structural plan view of the TD-10419 building (dimensions in mm)

Typical school building has a RC moment-resisting frame load carrying system in the longitudinal direction and dual system in the transverse direction. There is no irregularity in its structural system. The only deficiency for this case study building can be the strong beam-weak column action. Due to the high dead and live loads on the slabs, deep beams were used in such kind of buildings. Typical beams are 300/400 x 800 mm^2.

Material Properties

For nonlinear analysis of the school building, material properties are taken from the blueprints of the case study. Concrete strengths is taken as 16 MPa as given in its blueprint. Grade 220 MPa reinforcement is used for both longitudinal and transverse direction. The yield strength of both longitudinal and transverse reinforcement is taken as 220 MPa. Strain-hardening of longitudinal reinforcement has been taken into consideration and the ultimate strength of the reinforcement is taken as 330 MPa [14]. Ultimate strain for Grade220 steel is taken 0.18 as given in TS500 [15]. Typical transverse reinforcement given in design drawings is ϕ8 with 150 mm spacing for columns.

MODELING OF THE CASE STUDY BUILDING

Three dimensional model of the typical design was prepared in the SAP2000 environment [12]. For nonlinear analysis, member sizes and reinforcements in the typical design were used. All members were modeled as given in the template design. Flexural elements for beams, beam-column elements for columns and shear-walls and rigid diaphragms for floors were employed for modeling the structural components. Infill walls are not modeled in the analysis of the buildings.

Pushover analyses were conducted to determine the capacity curves for both orthogonal directions. Then modal properties were determined consistently, which match to the initial linear part of the bilinear representation of the capacity curves. Beam and column elements were modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges. Nonlinear flexural characteristics were defined by moment-rotation relationships of plastic hinges assigned at the member ends. Column capacities were calculated from the three dimensional axial force – bending moment interaction diagrams. Nonlinear behavior of shear walls is modeled using FEMA – 356 guidelines [6]. A typical moment-rotation relationship for frame members is shown in Fig 2. The segment AB, representing initial linear behavior, is followed by the post-yield behavior of BC. Point C corresponds to the ultimate strength. The drop in strength from C to D represents the beginning of failure in the member.

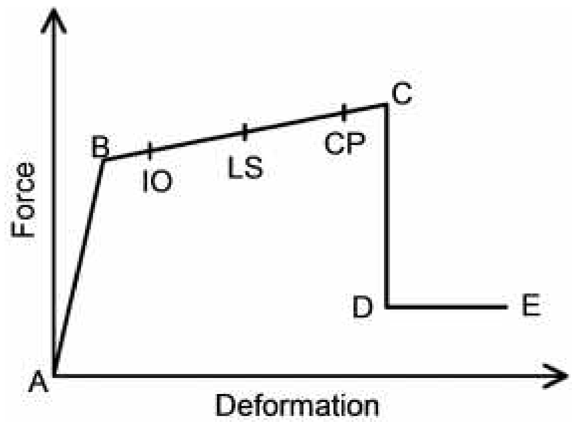


Figure 2. Idealized Force-Deformation relationship of a frame member end

Mander model was used for unconfined and confined concrete while typical steel stress-strain model with strain hardening for steel [5] was implemented in moment-curvature analyses. The points B and C on Fig 2 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 [2]; 0.5EI and 0.70EI for beams and columns, respectively [6]. 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. [11], given in Equation 1, and (3) the longitudinal steel reaching a tensile strain of 60% of ultimate strain capacity that corresponds to the monotonic fracture strain [11]. Decreased values (60%) are considered in order to reflect buckling of longitudinal bars during reversed loading cycles. Ultimate concrete compressive strain (ε_{cu}) is given as

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

(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. Plastic hinge length is used to obtain ultimate rotation values from the ultimate curvatures. Several plastic hinge lengths have been proposed in the literature [5, 9, 11]. In this study plastic hinge length definition given in Equation 2 which is proposed by Priestley et al. is used [11].

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

In Equation 2, L_p is the plastic hinge length, L is the distance from the critical section of the plastic hinge to the point of contraflexure, 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 in 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 [3, 13].

Pushover and Nonlinear Response History Analyses

Pushover curves of school building are obtained by using SAP2000, in two orthogonal directions. The lateral forces applied at mass center 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. The capacity curves of the building obtained from pushover analysis was approximated with bilinear curves using FEMA -356 and reduced to “equivalent” SDOF systems according to guidelines given in ATC-40 [2] and FEMA-440 [7]. Then these SDOF systems are subjected to nonlinear response history analysis by using ground motion records with the software BiSpec [8].

EARTHQUAKE RECORDS

One hundred ground motion records with different range of intensities are used for the study. All earthquake records were taken from PEER website [10].

PERFORMANCE EVALUATION

Performance evaluation is carried out according to TEC-2007 [17] regulations. 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 [6] and ATC-40 [2]. Pushover analysis data and criteria of TEC-2007 were used to determine global displacement capacity of the building corresponding to the performance levels considered. Displacement demand estimates obtained by nonlinear response history analyses of "equivalent" SDOF models are compared for IO, LS and CP displacement capacities. Performance level of the building is determined for each of the earthquake loading. Ratio of buildings exceeding given performance level to all buildings are determined for each earthquake. This value is assumed as probability of exceedance for the given performance level for the considered earthquake. If the earthquake is expressed with a ground motion indices (such as PGA or PGV), the probability of exceedance of a performance level for a certain ground motion indices is determined. When the cumulative probability of the exceedance is plotted against the considered ground motion indices, the fragility curve for the corresponding performance level is obtained.

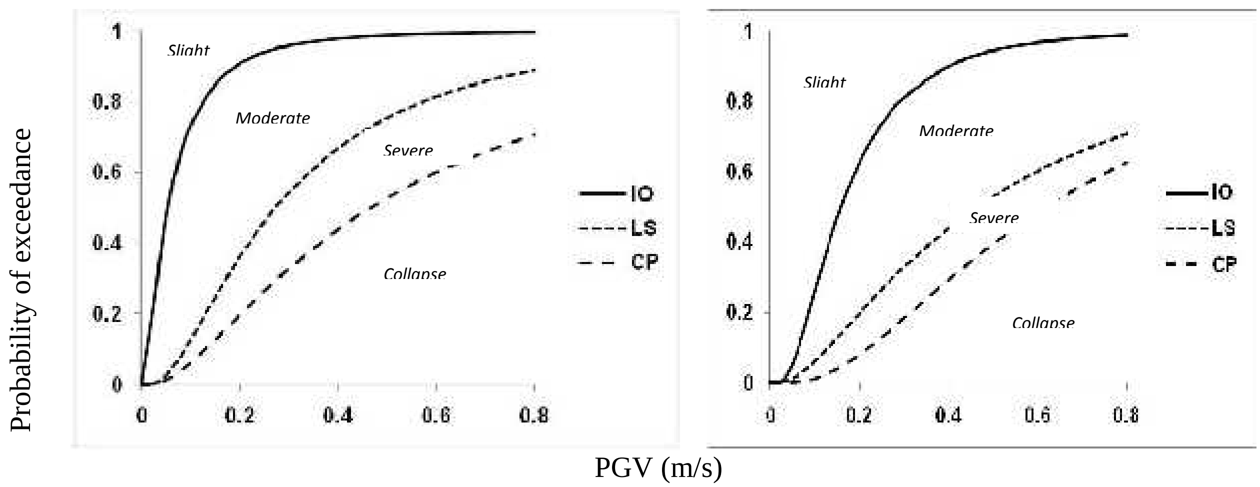
In this study, the cumulative probability of exceedance values for earthquake indices are defined as explained above. PGV values are selected as earthquake indices representing the ground motion intensity. The fragility curves are obtained by assuming log-normal distribution functions.

ANALYSES RESULTS

Inelastic time history analyses are deployed to derive the fragility curves of the selected template design. Computed maximum drift values by pushover analyses were supposed to represent the seismic performance of the school building. Using the damage limit state levels defined in TEC-2007, the exceedance probabilities of that particular fragility curve were calculated from the PGV versus maximum global drift scatters specific to each building.

Fragility curves are shown in Fig 3 for the template design. The three curves in each figure represent the probability of exceeding the IO (slight damage); LS (moderate damage) and CP (severe damage) limit states respectively.

Peak velocities of several past earthquakes have shown that between 0.40 and 0.70 m/s of PGV values could be attributed to the severe earthquakes. In order to make a prediction of damage level, this interval is accepted and an indicator of severe earthquakes. According to TEC-2007, school buildings are expected to satisfy IO and LS performance levels under design and severe earthquakes, corresponding to 10% and 2% probability of exceedance in 50 years. The probability of exceedance of performance levels for PGV values corresponding to PGA'a of (0.2g, 0.4g and 0.6g). Besides, CP performance level is added as a further detail.



a) *x- **direction***   **b) y- *direction***

Figure 3. Fragility curve of typical school building

Based on the analytical damage evaluation of school building, following observations can be made for the considered concrete and detailing quality:

- The level of probable damages for C16-S150 are: 95% probability to have slight damages; 75% probability to have moderate damages; 58% probability to have severe damages for *x*-direction, whereas these values are 90%, 50%, and 37% respectively for slight, moderate and severe damages on *y*- direction.

CONCLUSION

In this study, the vulnerability of a typical RC school building from Turkish practice is investigated. Mathematical model of the building is established by using outcomes of a detailed inventory study. Capacity curves of building are determined by pushover analyses conducted in two principal directions. The inelastic dynamic characteristics are represented by “equivalent” SDOF systems using obtained capacity curves of buildings. The nonlinear dynamic response history analyses results of the “equivalent” SDOF models with 100 acceleration records are evaluated. In light of these analyses results, fragility curves for different cases are determined. The considered cases are the building code as being modern (TEC-2007, 2007) or pre-modern (TEC-1975, 1975), lateral steel amount and detailing, and material quality.

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