Seismic Risk Prioritization of RC Public Buildings In Turkey

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SUMMARY
Over the past years, Turkey has made substantial progress toward mitigation of earthquake risks, through changes to the regulatory frameworks. Revisions were made to the building code, which now includes standards for seismic retrofitting. The Turkish Government intends to initiate studies for the dissemination of the public retrofitting program currently being implemented in Istanbul to selected high risky provinces throughout the country. The technical methodology for the prioritization of seismically vulnerable public facilities is presented herein. The methodology is mainly based on the lateral force demand versus base shear capacity. A form is developed that would include the required information and data to be filled in by the related provinces in order to create inventories and prioritization lists of buildings by province. The proposed building prioritization methodology has been calibrated with the survey results of several damaged public buildings after the recent earthquakes in Turkey.

Keywords: Seismic risk, prioritization, public buildings, technical vulnerability, social vulnerability

1. INTRODUCTION

A simple method has been developed for the seismic risk prioritization of reinforced concrete public buildings in Turkey. The lateral load capacity of the building is determined approximately in this method and compared with the lateral load demand of the earthquake at the building site. Then a performance index is derived from this comparison, which is the basis of risk prioritization. As built structural data and field data are required for the buildings that are selected for investigation. The proposed procedures are presented in detail, and then tested on several case studies for verification and calibration with reference to the results of detailed inelastic analysis procedures.

2. COMPONENTS OF PRIORITIZATION METHOD

A general flowchart of the procedure proposed for risk prioritization is shown in Figure 1. Basic elements of the procedures are explained in the following sections.

As-built structural system properties of a building should be determined in sufficient detail for seismic risk assessment. Then a simple structural simulation of the building is prepared, and seismic capacities and internal force demands are calculated under the defined earthquake excitation for seismic performance evaluation.

3. CONDITION ASSESSMENT

The main purpose of the condition assessment of an existing building is to determine its structural characteristics. Assessment studies are carried out before the earthquake and a knowledge database is developed for calculating the seismic performance of the building under an expected earthquake excitation. Since the objective is risk prioritization rather than a detailed performance assessment, data collection from for the building should be completed within a reasonable duration. Although this duration varies for different buildings, it should not exceed half a day with a team of two technicians for a moderate size public building. The knowledge collected from the building through condition assessment is essential in developing its analytical model, performing its seismic analysis and evaluating its seismic risk. Hence it is required to determine the structural system characteristics, material properties and construction details of the building. Destructive methods for determining material properties and detailing are not appropriate for risk prioritization.

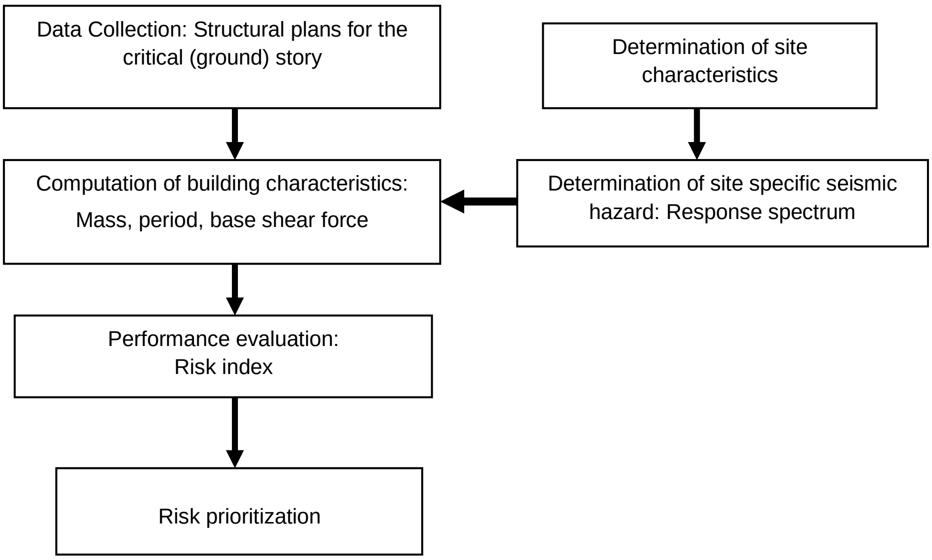

Figure 1. Flowchart of operations for risk prioritization in buildings

The other essential component in seismic risk assessment is the definition of seismic hazard at the building site. Site conditions and the GPS coordinates are the first set of data required in the site specific seismic hazard analysis. Soil type at the site should be determined in accordance with the soil types specified in the Turkish Earthquake Code (Ministry, 2007). Available geotechnical and geological maps or available field observations can be used for this purpose. Finally, seismic hazard at the building site related to the soil type and seismic hazard is defined in terms of spectral acceleration in accordance with the Turkish hazard map.

3.1. Building geometry

Building geometry as used in this study refers to dimensional and functional identification related to the structural system, foundation system and architectural form of the building. Identification of all member and component dimensions required in analytical modelling of the existing building is the main scope of geometric identification. Measurement and investigation procedures used for geometrical identification includes the preparation of as-built structural plans, and taking pictures from critical locations.

As-built structural plans consist of the structural system layout of critical story and cross section dimensions of members in the plan. A typical as-built structural plan drawing is shown in Figure 2. The locations and dimensions of columns, beams, load bearing walls, masonry infill walls, are indicated on the plan. Axis distances are measured and beam locations are determined. Total number of free standing stories is counted and critical story height is measured. Photographs of the exterior facades and critical interior locations of the investigated building are also very useful in making the risk evaluation of a building. These pictures may show the basic features of the structural system including overhangs, discontinuities, roof details and soft story formation.

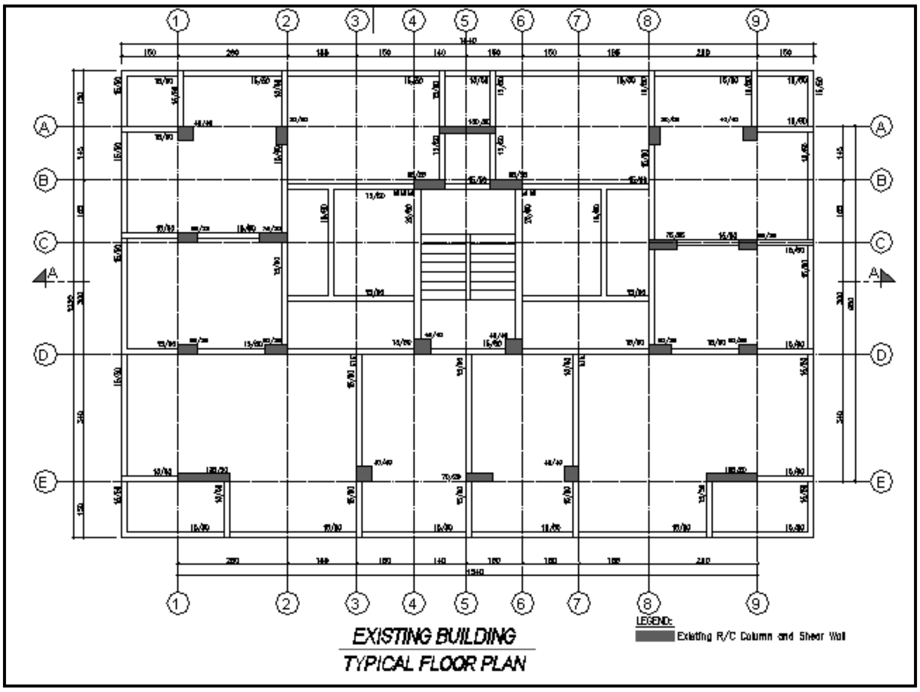

Figure 2. Typical structural as-built plan drawing

3.2. Reinforcement details of structural members

Reinforcement details required for reinforced concrete members are the ratios of longitudinal and transverse reinforcement, and the effect of corrosion observed at the reinforcing bars.

Available structural plans and non-destructive methods can be employed in determining the reinforcement details. However past experience indicates that most buildings do not have as built structural drawings. In this case, it is advised to use the minimum reinforcement ratios and detailing given in the codes effective during the time of construction. Removing the concrete cover for visual observation, or using metal scanners are time consuming, hence they do not fit to the scope of a prioritization study. These procedures are more appropriate for the condition assessment of prioritized buildings that are selected for retrofitting.

3.3 Concrete properties in an existing building

There are two basic non-destructive testing equipments for determining in-situ concrete strength. These are the Schmidt rebound hammer and ultrasound velocity equipment. Schmidt hammer is usually preferred in practice due to its low cost and practical use.

A reading by a rebound hammer actually indicates the surface hardness of concrete. This reading is then converted to equivalent concrete cylinder strength by using a calibration curve. The correlation between the hammer readings and laboratory tests on samples taken from the same buildings are shown in Figure 3, for 200 buildings surveyed in the Zeytinburnu sub province of Istanbul. It is observed that the correlation is reasonable despite some scatter in the measured values.

Since the primary objective in this study is risk prioritization, there is no need for taking concrete samples from the buildings and destroying several members. It is sufficient to take hammer readings from the columns at the critical story, and convert these readings to concrete cylinder strength by using the calibration curve of the hammer. Hammer readings should be taken from at least ¼ of the columns and concrete walls at the critical story, but not less than 3 columns or walls. Plaster will be removed from the surface, and at least 10 hammer readings will be taken from each member. Mean cylinder strength is used as concrete strength in capacity calculations.

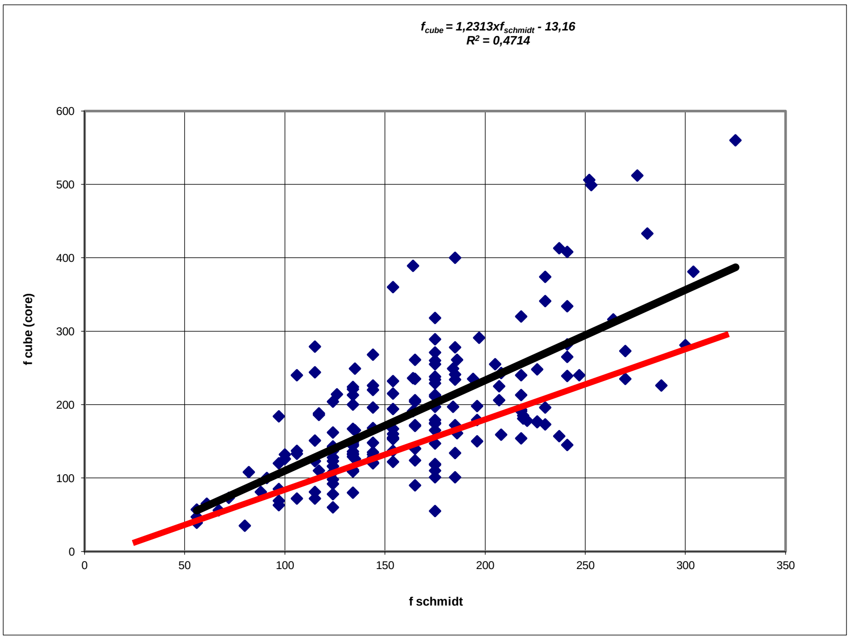

Figure 4. Correlation of hammer readings and laboratory tests on samples taken from 200 buildings at Zeytinburnu, Istanbul

3.4. Determining Reinforcing Steel Properties in an Existing Building

The variability in reinforcing steel is much less compared to concrete since steel is a serial industrial production. Steel class (S220, S420, plain bar, deformed bar, etc.) can be easily identified by removing the steel cover and observing the steel bar. However this is a tedious work which does not fit to the scope of risk prioritization. It is adequate to use the characteristic yield strength values that belong to the associated steel type given in design drawings in cross section capacity calculations (220 or 420 MPa). If the drawings are not available, then an assumption on steel type can be made by using expert opinion. S220 was the mostly used steel type in Turkey until 1980's.

4. IDENTIFICATION OF THE DYNAMIC CHARACTERISTICS OF A BUILDING

Building weight and vibration period should be determined for calculating the equivalent static lateral forces acting on a building. Building weight can be easily calculated from the as-built drawings. A distributed weight of w = 8-10 kN/m2 can be assumed safely for estimating the gravity loads, including the live floor loads.

Building period can be estimated quite accurately by using the number of floors and total building height. The vibration periods of 30 public buildings located in the high seismic zones of Turkey are calculated by rigorous procedures, and their variation with the total building height is shown in Fig. 5. Period-total height relationship displays less scatter compared to period-number of stories relation. Accordingly, building period is calculated from the approximate relationship $T=0.002*H^{2.25}$. This relation yields larger values compared to the expressions given in the seismic codes (Fig. 5). The reason is employing cracked section stiffnesses in deriving the proposed vibration period expression.

Linear elastic base shear force (V_{bs}) which acts on the building is calculated from Equation 1 by using the spectral acceleration and total building weight. The term S_a (T_1) in Eqn. 1 indicates the spectral acceleration value corresponding to the building period T_1.

$$V_{bs} = S_a (T_1)*W \tag{1}$$

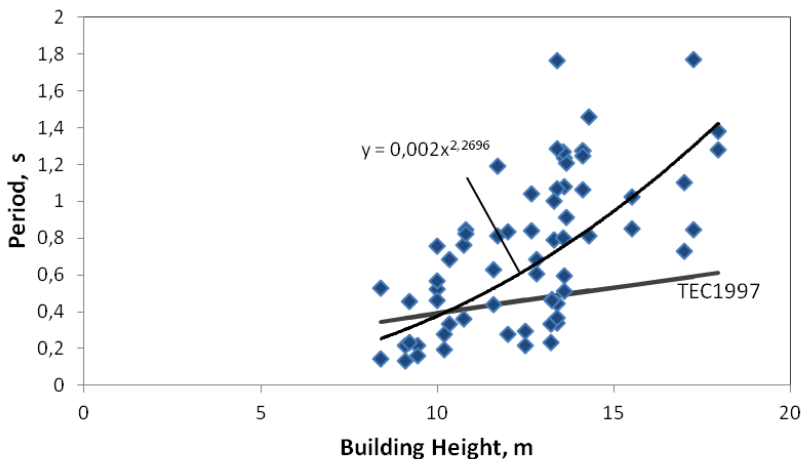

Figure 5. The relationship between the building period and the building height

5. RISK PRIORITIZATION METHOD

The proposed risk prioritization method is based on the comparison of lateral load resisting member capacities with the lateral load demand of the earthquake at the critical story of the building. Accordingly, the flexural and shear capacities of the columns at the critical story are calculated first. Structural plan, concrete strength, steel strength, column and wall dimensions, total longitudinal and transverse steel reinforcement ratios are required for calculating the column capacities. A data processing form for reinforced concrete public buildings is developed for this purpose in order to facilitate practical implementation of the prioritization methods. This form has to be filled for each building block. Tributary area of each column should be calculated from the plan drawing and entered to the associated cell in the table.

Column cross section dimensions should be determined separately for the x and y directions of the building and defined on the form. The column dimension in the x direction is defined by *b*, and the dimension in the y direction is defined by *h* (Fig. 6). For shear walls, the length is l_w and the thickness is *t*. It is assumed that the shear walls contribute to the system capacity only in their strong direction. If columns and walls are not aligned along the principal axes of the building, then the angle between the long direction and the x axis of the building should be defined. In this case both components of the member capacity along the principal axes are taken into account. Unreinforced infill walls which are continuous along the building height are also indicated on the data form, as well as on the critical story plan.

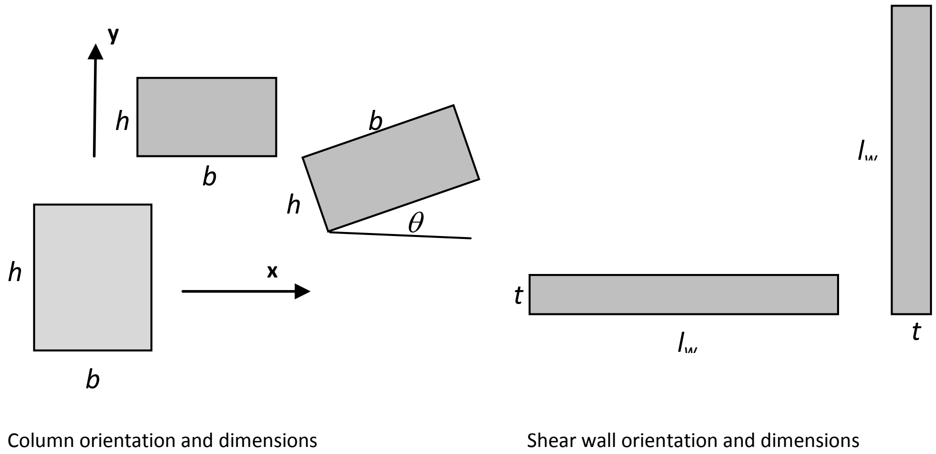

Figure 6. Definition of column and shear wall orientations and dimensions

The building risk is calculated in view of the information indicated on the building data form. The method which is developed for the seismic risk prioritization of reinforced concrete public buildings in Turkey is presented in the following section. It is based on the estimation of lateral force (story shear) capacity of the building at the critical story. Story shear capacity is calculated by considering the lateral load capacities of the shear carrying (vertical) members. The lateral load capacities of columns and shear walls are calculated in both principal directions, for two possible failure states: flexure and shear. The lower value in each direction is taken as the member lateral capacity. Total lateral capacity of the building in each principal direction is equal to the sum of the lateral capacities of all columns and walls in the critical story in that direction. Finally, the elastic lateral seismic load demand is divided by the capacity in each direction to calculate a risk index, which is analogous to the force reduction factor in seismic design.

5.1. Lateral load capacity of columns

Lateral load capacity of columns is controlled either by the flexural, or the shear failure mode. The lower value governs the column capacity. Lateral load capacity of a column corresponding to each failure mode is defined separately below.

Lateral Load Capacity of Columns in Flexure: Column flexural capacity depends on column axial load. Column axial loads are calculated under the gravity loads acting on the building. Column tributary area (A_{eq}) is required for calculating the column axial load. Column tributary areas are determined in view of the distribution of columns on the plan area. Tributary areas are determined by considering the geometric shape of plan area (vertical loads) by the columns. This is similar for the shear walls.

Column axial load (N_d) is calculated from Eqn. 2, where n_s is the number of unconstrained (free to vibrate) stories. When the tributary area A_{eq} is defined in square meters in Eqn. 2, N_d is obtained in kN.

$$N_d = A_{eq} \cdot n_s \cdot (8\ \text{kN/m}^2) \tag{2}$$

Column flexural moment capacities Mp which are dependent on N_d are theoretically calculated by conducting a section analysis. However this is time consuming and not practical for risk prioritization. Alternatively, column and shear wall flexural capacities can be calculated with sufficient accuracy from Eqn. 3.

$$M_p = 0.5 N_d \left(h - 1.2 \frac{N_d}{f_c b} \right) + 0.5 A_{st} f_y (d - d') \tag{3}$$

Here, h is the section depth, b is the section width, d' is the cover thickness, A_{st} is the ratio of longitudinal reinforcement, f_c is the concrete compressive strength and f_y is the yield strength of longitudinal reinforcement. All parameters should be expressed in consistent units, preferably SI. Moreover, $d=h-d'$ and d' can be taken as 3 cm in practice.

It is assumed that lateral load capacity of a column in flexure develops with the formation of plastic hinges at both ends. If the flexural capacity at top and bottom ends of a column are M_{ui} and M_{ai} respectively and the column clear height is h_s, then the column lateral load capacity in flexure is calculated from,

$$V_{yi} = (M_{ui} + M_{ai})/h_s \tag{4}$$

If flexural capacities of the member ends are calculated from Eqn. (3), then $M_{ui}=M_{ai}=M_p$.

Lateral Load Capacity of Columns in Shear: Column shear capacity V_r is composed of concrete contribution V_c and transverse shear reinforcement contribution V_s. V_r is calculated from Eqn. 5, where f_{yw} is the yield strength of transverse reinforcement, A_{sw} is the total area of transverse reinforcement, s

is the spacing of transverse reinforcement, and f_{ct} is the tensile strength of concrete (f_{ct} = 0.1 f_c can be assumed).

$$V_{ri} = \frac{A_{sw} f_{yw} d}{s} + 0.325 f_{ct} bd\psi \; ; \; \psi = 1 + 0.07 \frac{N_d}{bd} \qquad (5)$$

5.2 Lateral Load Capacity of the Critical Story

Shear capacity of a column calculated from Eqn. 5 is then compared with the lateral load capacity calculated for flexure from Eqn. 4. The lower value is accepted as the column lateral load capacity. If $V_{ri} > V_{yi}$, then $V_i = V_{yi}$, and if $V_{ri} < V_{yi}$, then $V_i = V_{ri}$.

Story lateral load capacity is the sum of the lateral load capacities of all columns and shear walls in that story. Accordingly, the lateral load capacity of the critical story is given by Eqn. 6 below.

$$V_b = \sum V_i \qquad (6)$$

5.3. The contribution of unreinforced masonry infill walls to the story lateral load capacity

The effect of URM walls on the story lateral load capacity is important in preventing the total collapse of seismically deficient concrete frame buildings during an earthquake. This effect depends on the existence of imperforated walls continuous along the building height. A relationship was obtained between the base shear capacity and the area of URM walls for typical reinforced concrete buildings in Turkey (Yakut, 2004), shown in Fig. 7. The relationship in Fig. 7 expresses the ratio of base shear capacities of buildings with and without URM infill walls, plotted against URM wall area normalized with the total floor area. Hence, building lateral load capacity (base shear capacity) calculated from Eqn. 6 is modified to account for the effect of URM infills by employing Eqn. 7. In Eqn. 7, A_{sw} is the total area of URM infill walls in the critical story continuous along the building height and A_{ft} is the sum of building floor areas.

$$V_{bw} = V_b \left(46 \frac{A_w}{A_{tf}} + 1\right) \qquad (7)$$

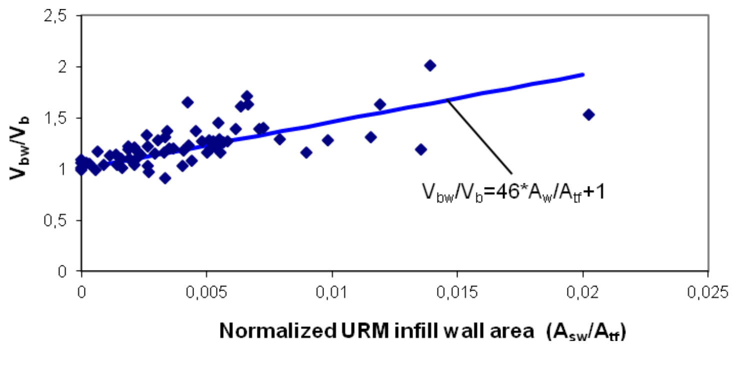

Figure 7. The effect of URM infill walls on the base shear capacity of buildings

5.4. Risk index

The risk index is calculated by using a force based approach. The risk index (*RI*) for a building is obtained by dividing the elastic base shear demand (Eqn.1) by the story lateral load capacity calculated from Eqn. 6, and further modified by Eqn. 7 where necessary. The larger of the *RI* value calculated for each principal direction of the building is used in risk prioritization.

$$RI = V_{bs}/V_{bw} \qquad (8)$$

An *RI* value less than 2.0 indicates that the building has an acceptable performance and its seismic risk is low. If *RI* value is between 2 and 4, the building may sustain some damage, however the collapse risk is low. *RI* larger than 4 indicates that the building has high risk. Since *RI* is a single parameter that expresses seismic risk, a risk prioritization can be made by using the *RI* values for a group of buildings. The *RI* values for the group can be ranked from highest to lowest, and the building at the top is considered as the one with the highest risk.

6. CASE STUDIES

In order to show the application of the proposed prioritization method explained and to test its validity, four public buildings were investigated. Properties of these buildings are summarized in Table 1. All case study buildings were modelled in detail and their pushover analysis were carried out to obtain their capacity curves. Performance assessment of the buildings was carried out by using detailed procedures (FEMA 356, 2000) and the results were compared with the proposed risk prioritization procedures.

Table 1. Properties of Case Study Buildings

Building no	No. of stories	Occupancy	Earthqauke Zone	Soil Class	Period (second)		Base Shear Capacity (kN)	
					T_x	T_y	V_{yx}	V_{yy}
BLD1	3	School	2	Z1	1.19	0.81	1050	1580
BLD2	4	School	2	Z2	0.79	1.00	3000	2400
BLD3	4	School	1	Z2	0.61	0.80	5300	4100
BLD4	4	Dormitory	1	Z2	0.84	0.56	3900	7000

Due to space limitations, only one case study will be presented in detail (BLD3).

6.1. Case study 3

The building named as BLD3 is a 4 story school building. The ground floor plan of the building is shown in Fig. 8. The vertical load resisting members of the building are composed of 40 columns. The building has 4 frames in x-direction (longitudinal) and 10 frames in y-direction. The concrete compressive strength of the building was determined as 7.0 MPa from concrete core tests. Based on the ferroscan and peeling of concrete, it was observed that plain bars were used as reinforcement. Its yield strength was taken as 220 MPa.

Cross section properties, tributary areas and axial loads of columns are calculated first. The longitudinal reinforcement ratio was taken as 1 percent of the cross sectional area for all members and the transverse reinforcement was assumed as 8 mm bars at 200 mm spacing. The height of ground story (critical story) is 3.15 m and the total height is 12.45 m. The building periods calculated from the relation shown in Figure 7 are 0.58 seconds in both x- and y-directions. The building is located in seismic zone 2 (PGA=0.3g) and its soil class was determined as Z2 (stiff soil). Ground floor area and total floor area of the building are 648 m2 and 2592 m2, respectively. Seismic risk of the building site was determined based on the spectrum given in Turkish Earthquake Code for 10 percent probability of being exceeded in 50 years. Accordingly, the spectral acceleration at the building period ($S_a(T)$) was determined as 1.03 g. The results of seismic risk prioritization for this building are summarized in Table 2. The building has no irregularity and its material/workmanship quality was evaluated as Poor.
Due to low concrete strength of the building, the capacity is governed by shear failure. According to the risk evaluation results presented in Table 2, the building possesses collapse risk in both directions. The building is classified as high risk.

Pushover analysis of the building was carried out in both directions, and the capacity curves obtained are shown in Fig. 9. The target roof displacements was also calculated by using the coefficient method proposed in the Turkish Earthquake Code (Ministry, 2007) under the design ground motion specified for the building site. All first story columns exhibit collapse performance at the target roof displacement in both directions.

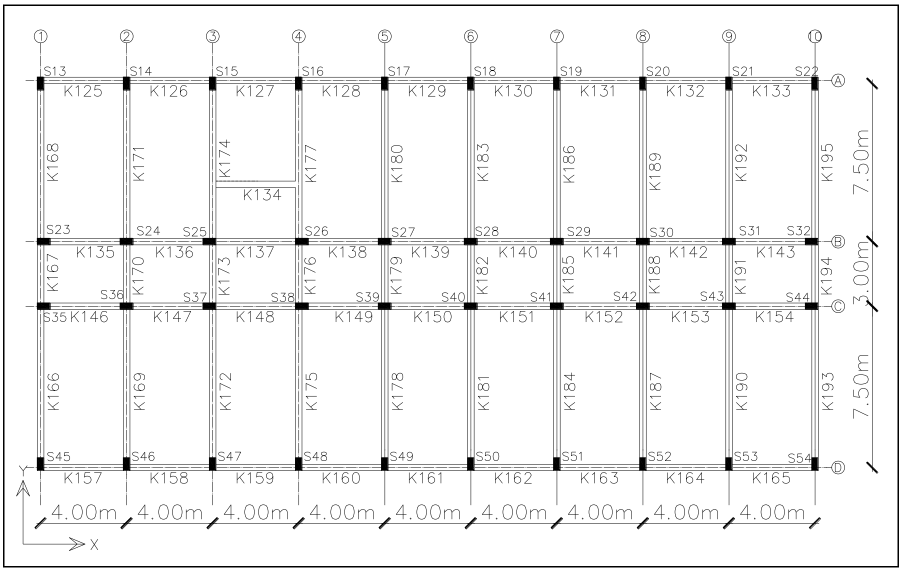

Figure 8. Ground floor plan of BLD3

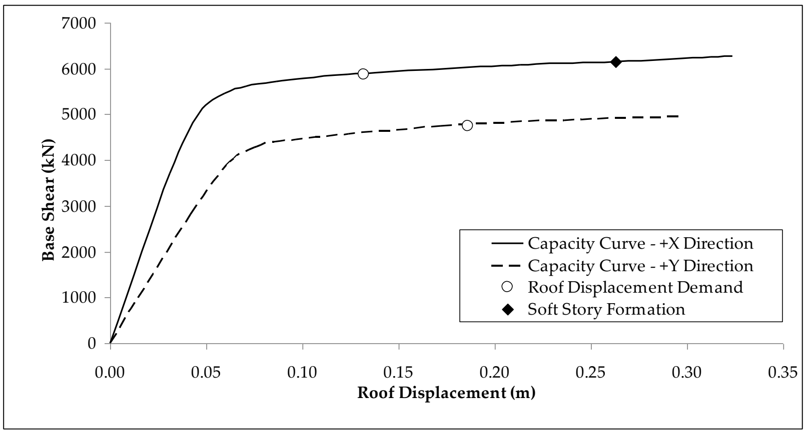

Figure 9. Pushover analysis results for BLD3

Table 2. Results for BLD3

	X-dir.	Y-dir.
V_{bs} (kN)	21366	
V_r (kN)	4031	4560
V_i (kN)	4278	4289
V_b (kN)	4031	4289
RI	4.99	4.98

6.2. Discussion of case studies

Detailed assessments of case study buildings were performed based on the results of pushover analyses. The evaluation of seismic risks based on these calculations is summarized in Table 3. These comparisons indicate that the proposed risk prioritization method yields results that are consistent with the ones obtained from performance analysis that employs pushover analyses. Risk prioritization of the case study buildings based on the proposed *RI* index is given in Table 4 where the most critical direction is considered for each building. This prioritization suggests that BLD4 has the highest risk whereas BLD2 has the lowest. The ranking given in Table 4 is in line with the results of detailed performance analysis (pushover) summarized in Table 3.

Table 3. Risk assessment results for case study buildings

Building	X-direction		Y-direction	
	Pushover	Prioritization Method	Pushover	Prioritization Method
BLD1	Life Safety	High Risk	Immediate Occupancy	Moderate Risk
BLD2	Life Safety	Low Risk	Life Safety	Moderate Risk
BLD3	Collapse	High Risk	Collapse	High Risk
BLD4	Collapse	High Risk	Collapse	High Risk

Table 4. Risk prioritization for case study buildings

Building	*RI*
BLD4	6.11
BLD3	4.99
BLD1	4.73
BLD2	2.18

7. CONCLUSIONS

It has been verified that the proposed risk prioritization methodology can be employed for the prioritization of public buildings for risk mitigation, specifically through seismic retrofitting. The procedure identifies the public buildings which should be considered with high priority in the implementation of a seismic retrofitting program. Buildings within the high priority class can be further classified according to their importance and social vulnerability, which is related to the services provided by the building and the population occupied in the building.

AKCNOWLEDGEMENT

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REFERENCES

Ministry of Public Works and Settlement (2007). Turkish Earthquake Code: Specifications for the Buildings to be Constructed in Disaster Areas, Ankara (in Turkish).

Sucuoğlu, H. and Karageyik, C. (2011). A Displacement-Based Approach for the Seismic Retrofitting of Medium Rise Non-Ductile RC Frames with Added Shear Walls, *Journal of Earthquake Engineering*, **15/6**, 959-968.

Yakut, A. (2204). Preliminary Seismic Performance Assessment Procedure for Existing RC Buildings, *Engineering Structures*, **26/10**, 1447-1461.