TECHNICAL NOTE

Seismic Risk Prioritization and Retrofit Cost Evaluation of Code-Deficient RC Public Buildings in Turkey

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A risk prioritization procedure is developed for deficient concrete public buildings within the scope of a seismic risk reduction program. The main purpose is identifying public buildings with high damage risk in a region for efficient retrofit investments. Regularity of structural systems and repeatability of deficiencies in public buildings provide opportunities for developing simple and reliable assessment procedures. The proposed procedure is based on calculating a risk index from the comparison of lateral load demand to lateral load capacity at the critical story of a building, and then prioritize the buildings in accordance with their risk index. Final decision for retrofitting is made with reference to the ratio of retrofitting cost versus demolishing and rebuilding cost. It has been shown on a sample of 70 retrofitted public buildings that the retrofitting cost ratio of deficient buildings is independent of risk level, age, height, floor area and concrete quality. [DOI: 10.1193/040513EQS092T]

INTRODUCTION

Several procedures have been proposed in the past for the seismic assessment of existing buildings. These methods range from visual screening (*FEMA 154* 2002, Sucuoğlu et al. 2007) to detailed assessment (*FEMA 310* 1998, JBDPA 2001, *ASCE 41* 2001). Visual screening or street survey procedures are usually carried out to determine the approximate seismic risk distribution in a large building stock and for estimating probable losses during a scenario earthquake. Detailed assessment, on the other hand, is carried out for buildings that are already considered for seismic retrofitting. These procedures are usually implemented in consequent tiers where the number of buildings investigated in each tier is reduced by screening out those with lower risk. ATC 3-06 (1978) is one of the earliest documents for determining the priorities for seismic intervention of a building inventory. Prioritization schemes for the seismic rehabilitation of hospitals in the United States specifically consider post-earthquake functionality (Holmes 2002). The New Zealand Society for Earthquake Engineering issued guidelines for assigning priorities and timescales for seismic retrofit (NZSEE 2003). Grant et al. (2007) developed a procedure for determining the priorities in the implementation of a seismic rehabilitation program for school buildings in Italy. Sucuoğlu et al. (2007) have

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developed a risk prioritization procedure for code-deficient residential concrete buildings in Istanbul, which is applied to 125,000 buildings located in close proximity to the Marmara fault. Building data are collected by visual screening in this procedure.

All risk assessment procedures are inherently regional in nature because the types of seismic deficiencies strongly reflect regional construction practices. Hence a procedure developed for a specific region or country by utilizing local data cannot be adopted in another region where the fundamental characteristics of deficient buildings may be quite different.

The method proposed herein is a single-tier procedure for ranking those public buildings in the order of their seismic risks within a province of Turkey, for the purpose of efficient retrofit investment decisions. Assessment and evaluation studies are carried out only at the critical story of a building. Public buildings in Turkey, especially the school buildings, are quite regular in structural framing, both in elevation and plan. Hence the critical story is usually the ground story, where maximum story shear demands occur. Furthermore, the deficiencies in a building are systematically repeated from one story to another. These typical characteristics are utilized to reduce the required field data and to simplify the analytical procedures in determining the risk priority of public buildings. As built structural data and field data are required only for the vertical elements of the critical story concerning columns and infill walls.

The proposed procedure is presented in detail, and then tested on several case studies for verification and calibration with reference to the results of more rigorous nonlinear static analysis procedure. Finally, a simple retrofit feasibility criteria is suggested, based on the ratio of initial retrofit investment cost to the cost of demolishing and rebuilding.

GENERAL CHARACTERISTICS OF CODE-DEFICIENT PUBLIC BUILDINGS

Most older public buildings in Turkey constructed before the 1990s are three- to six-story reinforced concrete frames with concrete slabs. Their framing systems are quite regular, especially in the school buildings that dominate the public building stock (see Figure 4). If large hospital buildings are excluded, this regularity usually prevails in all public buildings. Concrete quality is usually low, between 10–15 MPa. Plain reinforcing bars with nominal yield strength of 220 MPa are typical. Gravity loads dominate design; hence beams are stronger than the columns around a joint. Critical end regions of beams and columns are not confined; lateral reinforcement is basically provided for shear. Frame bays are infilled with unreinforced clay masonry walls at the exterior frames and at interior space separations. These buildings possess brittle seismic response confirmed by the past earthquakes consistently. Masonry infill walls contribute to lateral resistance until they completely fail.

The prioritization procedure suggested herein takes into account all of these common characteristics for minimizing both the fieldwork and analytical assessment.

BUILDING-SPECIFIC DATA

Building data includes basic structural information, including geometry, material properties, and information on seismic intensity defined for the associated seismic zone provided by the Turkish Seismic Zone Map (1996). Since the objective is risk prioritization rather than a detailed performance assessment, a data collection form 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-sized public building.

BUILDING GEOMETRY

Architectural plans of the public buildings are generally available at the public archives. The locations and dimensions of columns, beams, and masonry infill walls are indicated on the architectural plan. In this case, it is required to confirm the information on architectural plan drawings at the critical story during the survey of the building and to correct these drawings if there is inconsistency. If architectural plans are not available, then an as-built structural plan of the critical story is prepared in the field. Total number of freestanding stories is also counted and critical story height is measured. The critical story is usually the ground story where story shear is at a maximum in buildings with uniform elevation.

REINFORCEMENT DETAILS AND MATERIAL PROPERTIES

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 are valuable in determining the reinforcement details. However, past experience revealed that most buildings do not have as-built structural drawings. In this case, it is advised to assume type S220 steel ($f_{yc} = 220\,\text{MPa}$) and use the minimum reinforcement ratios and detailing for columns given in the codes effective during the time of construction. Removing the concrete cover for visual observation or using metal scanners is time-consuming; as such, they do not fit into the scope of a prioritization study. These procedures are more appropriate for the condition assessment of prioritized buildings that are selected for retrofitting. However, longitudinal reinforcement can be checked with metal scanners at the critical story in at least one typical column with most repeated section geometry.

In-situ concrete strength can be quickly determined by two non-destructive procedures. These are rebound hammer testing and ultrasound velocity measurement. Rebound 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 strength by using a calibration curve. The correlation between hammer readings and laboratory tests on samples taken from the same buildings are shown in Figure 1 for 200 buildings surveyed in Istanbul. It is observed that the correlation is reasonable despite some scatter in the measured values. The correlation coefficient is calculated as 0.60.

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 one-fourth of the columns at the critical story, but not less than three columns. These minimum numbers of hammer readings are consistent with the core sample requirements in the Turkish code (TEC 2007). Plaster will be removed from the surface, and at least ten hammer readings will be taken from each member. Mean cylinder strength is used as concrete strength in capacity calculations.

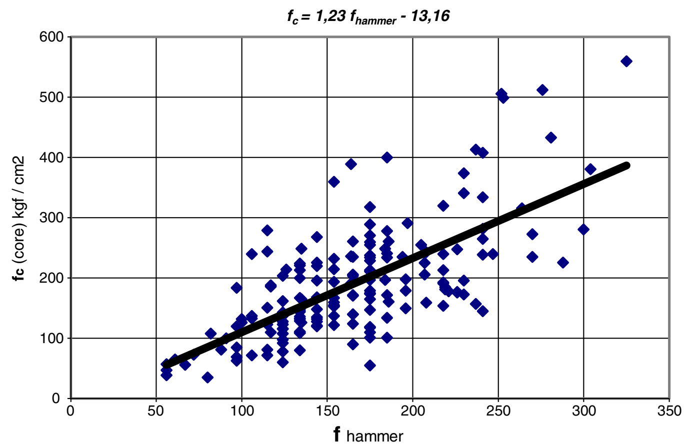

Figure 1. Correlation of hammer readings with the laboratory tests on samples taken from 200 buildings in Istanbul.

SEISMIC INTENSITY

Local soil conditions and the GPS coordinates are the first set of data required in determining the seismic intensity. Soil type at the site can be determined in accordance with the soil classes specified in the Turkish Earthquake Code (Ministry 2007). Available geotechnical and geological maps or available field observations can be used for this purpose. Seismic hazard (PGA) is obtained from the Turkish Hazard Map by using the building GPS coordinates for determining the relevant seismic zone. Finally, seismic intensity at the building site for fundamental vibration period is calculated by using the design spectrum in the Earthquake Code, which is formulated in terms of local soil class and seismic hazard.

RISK PRIORITIZATION METHOD

The proposed risk prioritization method is based on the comparison of lateral load demand of the earthquake with the sum of lateral load capacities of resisting members at the critical story of the building.

LATERAL LOAD DEMAND

Building weight and vibration period are required for calculating the equivalent static lateral loads acting on a building. Building weight W can be easily calculated from the as-built drawings. A distributed weight of $w = 8 - 10$ kN/m^2 can be assumed safely for estimating the gravity loads, including the live floor loads.

Building period can be estimated approximately by using the total building height. Vibration periods of 33 public buildings located in the severe seismic zones of Turkey are calculated by rigorous procedures, and their variation with the total building height is obtained as shown in Figure 2 (Kalem 2010). The results shown in Figure 2 are obtained from analyses of buildings without considering the masonry infill walls. Accordingly, a building's natural period is calculated from the approximate relationship $T_1 = 0.002 \times H^{2.27}$. This relation

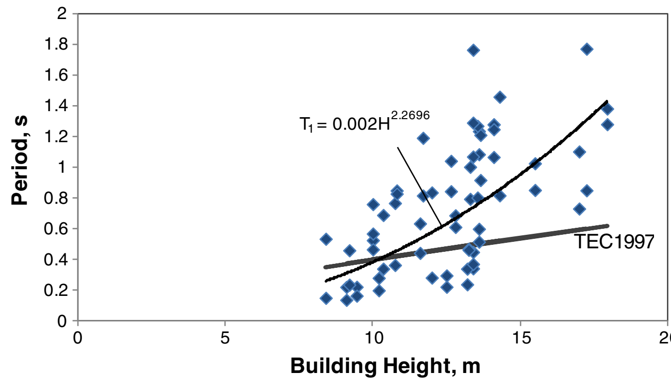

Figure 2. Relationship between building period and building height.

yields larger values compared to the expressions given in the seismic codes (Figure 2) due to the use of cracked section stiffnesses in deriving the proposed vibration period expression. The period equation yields smaller values when compared to the ones given in the literature for similar buildings in other countries (Massi and Vona 2010). This is due to different construction practices in Turkey and other countries, as well as using relatively larger modulus of elasticity as suggested by the Turkish Code.

Linear elastic story shear force (V_S), which acts at the critical story of the building, is calculated from Equation 1 by using the spectral acceleration and total building weight. V_S is usually the base shear force since the critical story is likely the ground story for most buildings. $S_a(T_1)$ in Equation 1 is the spectral acceleration corresponding to the building natural period T_1, and I is the building importance factor which is 1.5 for hospitals and emergency facilities, 1.4 for schools and 1.0 for ordinary public buildings.

$$V_S = S_a(T_1) \times I \times W \tag{1}$$

LATERAL LOAD CAPACITY OF THE CRITICAL STORY

Lateral load capacity of columns is controlled either by flexure or shear failure mode. The lower value governs the column capacity.

Column flexural capacity depends on column axial load. Column axial loads are calculated under gravity loads acting on the building. Column tributary area (A_{eq}) is required for calculating the column axial load. They are determined in view of the distribution of columns on the plan area. Column axial load (N_d) is calculated from Equation 2, where n_s is the number of stories above the critical story.

$$N_d = A_{eq} \times n_s \times w \tag{2}$$

Column flexural moment capacities, M_p, can be calculated accurately 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 Equation 3 (Ersoy et al. 2008).

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

Here, h is the section depth, b is the section width, d' is the cover thickness, A_{st} is the area of longitudinal reinforcement, f_c is the concrete compressive strength, and f_y is the yield strength of longitudinal reinforcement. $d = h - d'$ 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. This is an upper-bound value; however, it is considered appropriate since columns are weaker than beams in deficient public buildings. Accordingly, if the flexural capacity at both ends of a column i is M_{pi} and the column clear height is h_s, then the column lateral load capacity in flexure is calculated from:

$$V_{yi} = 2M_{pi}/h_s \tag{4}$$

If there are short or captive columns in the building, then h_s should represent clear height in these columns.

Column shear capacity, V_r, is composed of concrete contribution, V_c, and shear reinforcement contribution, V_S. V_r is calculated from Equation 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 ($f_{ct} = 0.1f_c$ can be assumed). If as-built drawings are not available, then 8 mm stirrups with 250 mm spacing along the column height can be assumed for typical reinforced concrete frames designed for gravity in Turkey.

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

V_{ri} is then compared with V_{yi}, calculated for flexure from Equation 4. The lower value is accepted as the column lateral load capacity. If $V_{ri} > V_{yi}$, then $V_i = V_{yi}$. 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 in that story. Accordingly, the lateral load capacity of the critical story is given by:

$$V_b = \sum V_i \tag{6}$$

The effect of unreinforced masonry (URM) walls on the critical story lateral load capacity is important in seismically deficient concrete frame buildings during an earthquake. A relationship was obtained between the base shear capacity and the normalized area of URM walls for typical reinforced concrete buildings in Turkey (Yakut 2004), which is shown in Figure 3. Hence building lateral load capacity (base shear capacity) calculated from Equation 6 is modified in Equation 7 in order to account for the effect of URM infills on the lateral load capacity of critical story. In Equation 7, A_{sw} is the total area of URM infill walls in the critical story, and A_{ft} is the total floor area.

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

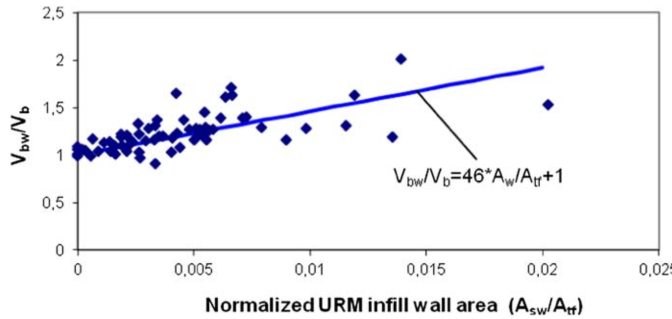

Figure 3. The effect of URM infill walls on the base shear capacity of buildings.

Equation 7 is evaluated for each orthogonal direction x and y, separately.

The possibility of soft story failures and torsional irregularity effects are ignored in the proposed procedure for maintaining simplicity. Although they are quite unlikely in public buildings, they may exist in some public buildings.

RISK INDEX

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

$$RI = V_S / V_{bw} \tag{8}$$

An *RI* value lower than 2.0 indicates that the building has an acceptable performance, and its seismic risk is low. If an *RI* value is between 2 and 4, the building may sustain some damage, however 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.

RI is analogous to the load reduction factor *R* employed in the Turkish seismic design code, which ranges from 4 to 8. The *R* factor also includes overstrength. Typical values of overstrength for ordinary Turkish buildings are around 1.4–2.3 (Yakut 2008). When the *R* factor given in the code is expressed only for ductility, without the average overstrength value of nearly 2.0, then range of *RI* would be 2.0–4.0. Therefore, the risk index is an indication of the code compliance of a given building. Thus, the ranges selected for *RI* are based on the code limits and case study results.

CASE STUDIES

In order to show the implementation of the proposed prioritization method and test its validity, six public buildings are investigated. The general properties of these buildings are summarized in Table 1. Four of these buildings are existing buildings, and two had sustained damage from past earthquakes. Periods and base shear capacities were determined from

Table 1. Properties of case study buildings

Building	Number of stories	Type	Earthquake zone (TEC 2007)	Soil class (TEC 2007)	Period (s)		Base shear capacity (kN)	
					T_x	T_y	V_{bx}	V_{by}
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
BLD5	4	City Hall	1	Z2	0.80	0.85	–	–
BLD6	5	Public office	1	Z2	0.70	0.65	–	–

eigenvalue and pushover analyses, respectively. Four buildings have four stories, one building has three stories, and the last one has five stories. The first two buildings are in seismic zone 2 (PGA = 0.3 g), and the last two buildings are in seismic zone 1 (PGA = 0.4 g). Buildings BLD1 and BLD2 have varying percentages of infill walls (0.2% to 0.53%) in the two directions, whereas BLD3 and BLD4 do not have infill walls that are considered to contribute to lateral load resistance. Performance assessment of the first four buildings is carried out by using the nonlinear static procedure given in *ASCE 41* (2001), and the results are compared with those of the proposed risk prioritization procedure. The buildings that were damaged from previous earthquakes were assessed by the proposed prioritization procedure only.

In pushover analyses, columns and beams were modeled as frame elements with lumped hinge properties assigned at member ends. Capacity spectrum method was used to determine the target displacement. The results for the third building BLD3 are presented in detail, and the other results are summarized.

The ground floor plan of the four-story building BLD3 is shown in Figure 4. Concrete compressive strength of the building was determined as 7.0 MPa from concrete core tests. Plain bars were used as reinforcement where the yield strength was 220 MPa. Structural drawings and details were available. All columns have 60 × 30 cm rectangular dimensions, with the orientations shown in the figure. The height of ground story (critical story) is 3.15 m and the total height is 12.45 m. The natural periods calculated from the relation shown in Figure 2 are 0.612 s in both x- and y-directions. The building is located in seismic zone 1 (PGA = 0.4 g) and its soil class was determined as Z2 (stiff soil). Ground floor area and total floor area of the building are 648 m^2 and 2,592 m^2, respectively. The seismic risk of the building site was determined based on the spectrum given in Turkish Earthquake Code for 10% probability of being exceeded in 50 years, and $I = 1.4$. 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. Due to low concrete strength of the building, the capacities of columns are 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.

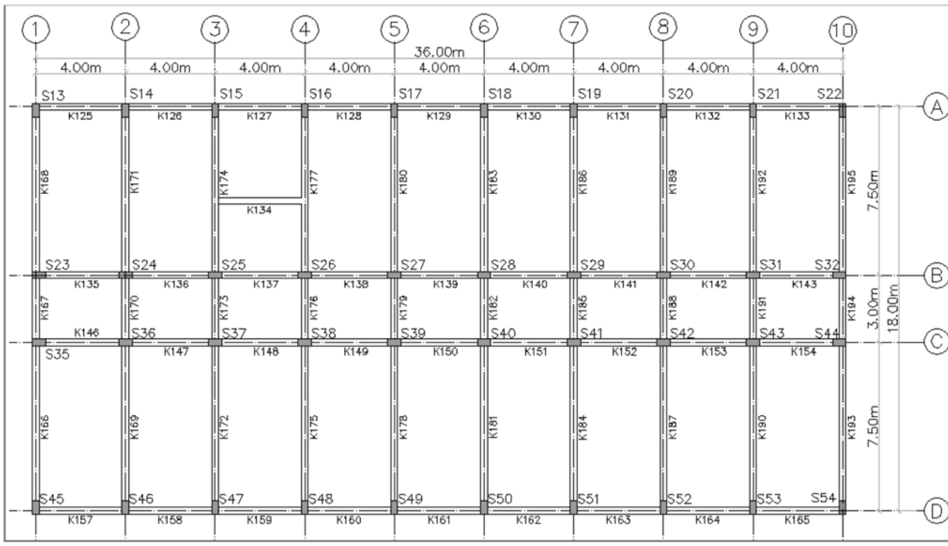

Figure 4. Ground floor plan of BLD3.

The comparative results of risk assessment for the first four buildings are presented in Table 3. These comparisons indicate that the proposed risk prioritization method yields results that are consistent with nonlinear static procedure. For the damaged buildings, the prioritization procedure yields moderate risks for both buildings that experienced moderate damage during the past earthquakes. For BLD5, observed damage concentrated on the ground floor where one column suffered heavy damage, four were moderately damaged and one was lightly damaged out of 23 columns. In BLD6, some columns in the ground floor suffered diagonal cracks.

ECONOMICAL ASSESSMENT OF SEISMIC RETROFITTING

When structural retrofitting of public buildings is required due to seismic safety considerations, the crucial decision is whether to demolish and rebuild a new building, or retrofit with a conventional retrofit technique. Life cycle cost analysis, which accounts for all costs that can occur during the life of a building can be employed to value buildings under consideration for these two options. If different cost options are to be compared for a public building, or if two different buildings are to be compared from a cost point of view, either present-value costs or annual equivalent-value costs are calculated using an appropriate

Table 2. Results for BLD3

Direction	V_S (kN)	V_r (kN)	V_i (kN)	V_b (kN)	*RI*
X	21,366	4,031	4,278	4,031	5.30
Y	21,366	4,560	4,289	4,289	4.98

Table 3. Risk assessment results for case study buildings

	X-direction		Y-direction	
Building	Pushover/ observed damage	Prioritization	Pushover/ observed damage	Prioritization
BLD1	Life safety	Moderate risk ($RI = 3.73$)	Imm. occupancy	Moderate risk ($RI = 2.40$)
BLD2	Life safety	Low risk ($RI = 1.71$)	Life safety	Moderate risk ($RI = 2.18$)
BLD3	Collapse	High risk ($RI = 5.30$)	Collapse	High risk ($RI = 4.98$)
BLD4	Collapse	High risk ($RI = 6.11$)	Collapse	High risk ($RI = 5.00$)
BLD5	Moderate damage	Low risk ($RI = 1.86$)	Moderate damage	Moderate risk ($RI = 2.88$)
BLD6	Moderate damage	Low risk ($RI = 1.68$)	Moderate damage	Moderate risk ($RI = 2.70$)

discount rate for comparison. Hence, for the assessment of seismic retrofitting with alternative rebuilding, all costs and all revenues should be considered for both options and then they should be compared by their present values or annual equivalent values (Hopkins et al. 2006).

Financial analysis is based on revenues versus costs. On the revenue side, it may be assumed that the building is rented throughout its lifetime and the rents thus collected are taken as revenues. It has to be considered here that retrofitting extends the remaining lifetime of an existing building, and makes it closer to a new building. The rental value of a retrofitted building is expected to be somewhat lower than an equivalent new building. For public buildings however, there is no intention for such commercial use. Therefore the revenues of "rebuilding" and "retrofitting" can be considered the same.

In addition to revenues, there are benefits from both options. When the rebuilding option is considered, the benefits are increased service capacity of the building (with changes in architectural layout), increased remaining life of the building, reduced maintenance costs during its lifetime, and continued services during the emergency phase after an earthquake. For the retrofitting option, regardless of the alternative methods of retrofitting implemented, the benefits are almost the same as those of rebuilding except the service capacity of the building, which will remain the same. A building's life is controlled usually by the economic lifetime of architectural and mechanical services and equipment (exterior facades, heat insulation, plumbing, electricity and lighting system, heating, air conditioning, etc.) rather than the structural system. Since these services are completely renewed during retrofitting, there is no significant difference between rebuilding and retrofitting as far as the benefits are considered.

On the cost side, the costs for the two options can be listed separately. For demolish-and-rebuild, these are the construction cost (during a construction period of two years), demolishing cost (initial), relocation cost (initial), and temporary settlement cost during construction (during a construction period of one year). On the retrofitting side, there are retrofitting costs including structural and nonstructural costs (during a construction period of six months), relocation cost (initial), and temporary settlement costs during the retrofit construction (during a construction period of six months).

It is discussed above that rebuilding and retrofitting are almost equal on the revenue and benefit sides. On the cost side, the initial costs of rebuilding and retrofitting dominate both options because demolition is added usually to the construction cost in rebuilding. Furthermore, the relocation cost is identical for both options, and temporary settlement costs do not matter for public buildings since the related services of the building are transferred to another public building during construction, rather than renting another building temporarily. It is also not realistic to evaluate the life-cycle costs for these items during such short construction phases of one year versus six months. Therefore, the major decision for public buildings is based on the comparison of total investment costs for rebuilding and retrofitting. Public administrations prefer such a simple feasibility approach since the comparative results are directly related to the required investment that should be allocated from the future budget.

COMPARISON OF THE INITIAL INVESTMENT COSTS FOR REBUILDING AND RETROFITTING

The ratio of the initial retrofit cost to the cost of rebuilding for public buildings, obtained from previously completed projects gives valuable information on the feasibility of retrofitting. Investigation of the data indicates that the average ratio is 20% for school buildings and other simple public buildings; however, it has large variations for major hospital buildings. A mean ratio of 33% has been obtained for retrofitting the moderately damaged buildings in Adana and Ceyhan after the 1998 Ceyhan earthquake (Bal et al. 2008).

Considering that the majority of public buildings that sustain significant seismic risk are either school buildings or simple building facilities, it has been decided to evaluate the correlation of the initial cost ratio (retrofit cost to the rebuilding cost) with several characteristics of the investigated buildings that can be considered to affect the cost of retrofitting significantly. These building characteristics are the risk index of the building determined by the technical prioritization procedure, mean concrete strength which is an indication of construction quality, size of the building both in the number of stories and the total floor area, and the age. The effects of these parameters on the cost ratio are discussed separately below.

If there is a significant correlation between the cost ratio and the risk index of a building, then it may be possible to decide directly on a threshold value of the risk index, *RI*: "Retrofit if the risk index is less than the threshold, and demolish and rebuild if it is larger." In Figure 5,

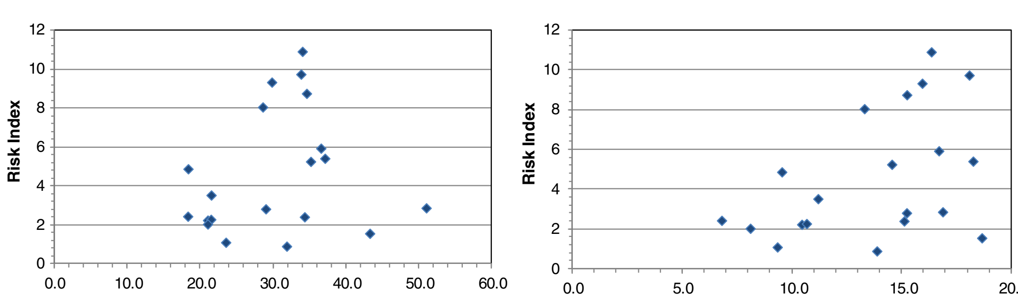

Figure 5. The variation of risk index with the total structural retrofit cost ratio.

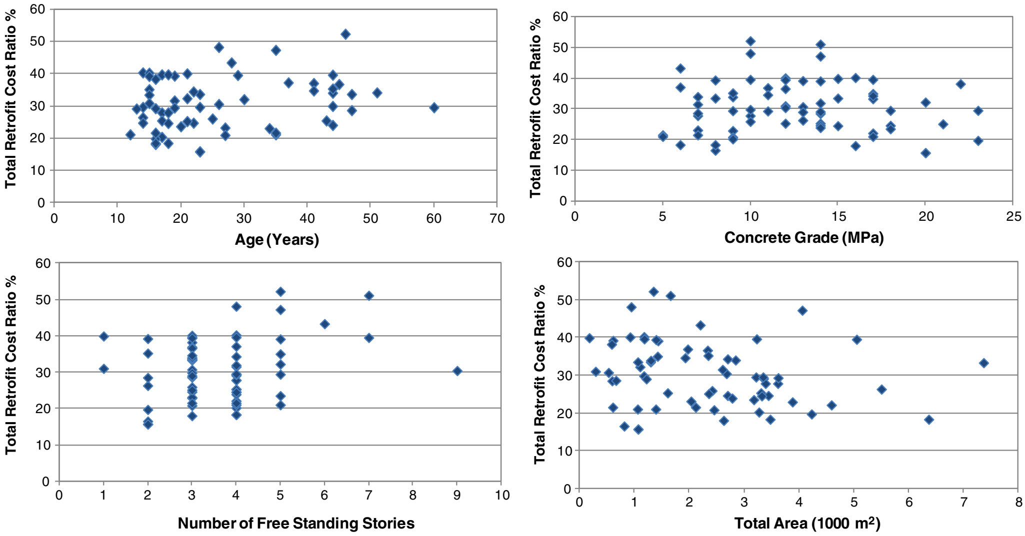

Figure 6. The variations of total retrofit cost ratio with the building age, concrete strength, number of stories and floor area.

the structural retrofit cost ratio and the total retrofit cost ratio are plotted separately with respect to the risk index for 20 retrofitted public buildings. It can be observed that the total retrofit cost ratio is not correlated with the risk index whereas structural retrofit cost ratio has a weak correlation. This result reveals that nonstructural costs of retrofitting mask the structural costs. Usually, structural costs are less than half the total cost of retrofitting. Furthermore, all structural components are upgraded to a target strength and ductility level regardless of the risk index during retrofit design. This is the main reason for the poor correlation between the risk index and the structural retrofit cost ratio.

A larger data set consisting of 70 retrofitted buildings has been developed to investigate the effects of mean concrete strength, size of the building, and the building age on the initial cost ratio. The average values of structural and total cost ratios for 70 buildings are 14.2% and 30.4%, respectively.

The effects of concrete strength, building age, building height and building size on the initial cost ratio are all evaluated graphically in Figures 6. These figures clearly indicate that there is no reasonable correlation between the total retrofit cost ratio and any of these building characteristics. There is a slight increase of the cost ratio with building age and the number of free stories, however these increases are not worth of consideration. There are only 7 cases in 70 where the total retrofit cost ratio exceeds 40%. Six of them are health centres, and the ratio is larger than 50% only in two cases. These cost ratios reveal that retrofitting is always feasible since retrofitting costs are much less than renewal costs.

CONCLUSIONS

The presented study reveals that simplified procedures developed for the risk prioritization of reinforced concrete public buildings are effective if the regularity of their structural systems and repeatability of the deficiencies are accounted for. The labor demand of the proposed prioritization procedure is minimal compared to rigorous assessment procedures, however the final decisions are sufficiently accurate. Moreover, retrofitting seems feasible in all cases constituting the database. Feasibility of retrofitting is only a function of the initial investment cost ratio whereas the risk level, age, size and concrete strength of the buildings has no influence on the retrofit cost.

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