DEVELOPING RETROFIT SOLUTIONS FOR THE RESIDENTIAL BUILDING STOCKS IN ISTANBUL

H. Sucuoglu[1], R. Jury[2], A. Ozmen[3], D. Hopkins[2], G. Ozcebe[1] and J. Kubin[3]

ABSTRACT

Feasible retrofitting solutions have been developed for selected residential mid-rise apartment buildings under high seismic risk in Istanbul. A feasible solution is considered as the optimal combination of cost, downtime, disturbance, technical applicability and social impact. The retrofit solutions include various combinations of strength and ductility enhancement at member and system levels. All solutions are classified into two major alternatives of external and internal retrofitting. External retrofitting mainly consists of exterior coupled shear walls attached to the building perimeter whereas the basic elements of internal retrofitting are concrete infilled shear walls. Secondary elements are also employed in both retrofitting solutions where necessary. They are basically the existing masonry partition walls enhanced by applying mesh reinforcement, FRP sheets or thin precast concrete panels on their surfaces. The seismic response from each alternative is calculated by simplified performance assessment procedures, and their acceptability is verified. Then their feasibility is tested against the "demolish and rebuild" alternative through a cost-benefit analysis. All solution alternatives have been reported to the building owners for their review. Exterior retrofitting has prevailed in most buildings as the feasible solution.

Introduction

Istanbul is the major candidate for wide scale retrofit applications, due to both the heightened odds of a severe earthquake along the Marmara segment of the North Anatolian Fault, and its immense building stock consisting of one million buildings where almost half is expected to be affected significantly from the foreseen Marmara earthquake. The scope of

[1]Professor, Dept. of Civil Engineering, Middle East Technical University, Ankara 06531, Turkey.
[2]BECA International Consultants Ltd, Wellington, New Zealand.
[3]PROTA Consultants Ltd, Farabi Sokak 7/5, Cankaya, Ankara 06660, Turkey.

this study is to conduct a pilot study in the Bakirkoy District Municipality of Istanbul, for testing the feasibility of retrofit investments on residential buildings in the region. Technical, economical and social aspects of retrofit implementations are envisaged as the crucial elements of feasibility. The final aim is to develop technically sound, innovative, economically feasible and socially acceptable retrofit methodologies that can be applied to the seismically vulnerable residential building stock in Istanbul, and possibly in other similar regions of Turkey. A sample of 373 residential apartment buildings was selected by the local authorities to represent the highly vulnerable building stock in the Bakirkoy region. Majority of the selected vulnerable buildings (295 out of 373) have 5 and 6 stories, whereas 44 have 3 and 4 stories and 34 have 7 to 9 stories. The retrofit techniques that are developed within the scope of the pilot study include various combinations of strength and ductility enhancement at both system and member levels. Their applicability and feasibility are tested and verified on a set of sample buildings.

The retrofit feasibility study conducted in Istanbul is presented in three consecutive papers in the 8NCEE. The first paper (Hopkins et al. 2005) presents an overview of the Bakirkoy project. The second paper presented herein focuses on assessing the seismic performances of the existing 373 buildings and developing retrofit alternatives as summarized above. The third paper (Johnston et al. 2005) reports on the social and economical components of the feasibility study.

Seismic Safety Assessment of Existing Buildings

Bakirkoy is a sub-provincial district in Metropolitan Istanbul, having a population of 150,000 and a building stock consisting of about 12,000 buildings, almost all of which are multistory reinforced concrete structures. Bakirkoy was chosen as the pilot region in the seismic retrofit feasibility project because of its proximity to the North Anatolian Fault, and the previous work done to assess its building stock for earthquake risks. In 2002, Bakirkoy Municipality undertook preliminary surveys of its residential building stock, including soil conditions and rapid assessment of each building. The survey enabled the authorities to identify approximately 3500 buildings as high earthquake risk. Owners of all 3500 buildings were invited to participate in the project, to be a part of detailed assessment. As a result of this process, 373 of the high risk buildings were selected as the subject of the retrofit feasibility study.

The field works carried out in each building for detailed seismic assessment is summarized in (Hopkins et al. 2005a). The objective of the detailed seismic assessment of 373 buildings in their existing state was to rank their seismic risks, and decide on the need or not for the buildings to be retrofitted.

Site-Specific Seismic Hazard

Seismic hazard in Bakirkoy was determined by a deterministic approach, which is based on a scenario earthquake of magnitude 7.2 along a segment of the North Anatolian Fault lying in the Marmara Sea basin. Then the attenuation functions developed for Turkey (Kalkan and Gulkan 2005) were employed along with the results of geotechnical surveys in the project field to determine the seismic hazard at each building site, expressed in the form of acceleration response spectra. The project site is in a distance range of 7 to 15 km from the North Anatolian

Fault. A sample of spectral shapes is shown in (Hopkins et al. 2005). Mean values of site specific response spectral ordinates were used in the seismic assessment of existing buildings.

Structural Assessment Procedure

A 3-D linear elastic modal spectral analysis procedure was employed, which is enhanced with the capacity principles in order to determine more realistically the column axial forces and potential locations where flexural capacities are exceeded. This procedure is called the "Capacity Control Method" (CCM), with the following basic principles.

a) Earthquake forces are not reduced with a response modification factor.
b) Cracked section moments of inertia are used. Accidental eccentricity is ignored.
c) Column axial forces are calculated as a combination of gravity and earthquake forces. However shear forces transmitted from beams to columns are limited with the shear forces corresponding to either the flexural, or shear capacity of beams.
d) Column-beam flexural capacity ratios (CBCR) are calculated at each joint. If CBRC>1.2 then beam ends have yielding potential, if CBRC<0.8 then column ends have yielding potential, otherwise all ends connecting to the joint have yielding potential.
e) If the demand to capacity ratio (DCR) at a potentially yielding frame member end or a masonry infill strut exceeds the DCR limit specified for a performance limit state, then that member performance is declared unacceptable. Otherwise it is acceptable. The DCR limits for columns at the collapse prevention limit state are given in Table 1 below.

Table 1. DCR limits for columns in the collapse prevention limit state

Ductile Columns		**DCR Limit for Collapse Prevention**
Confinement	$N / A_c F_c$	
Confined	≤ 0.1	7
Confined	≥ 0.4	5
Unconfined	≤ 0.1	5
Unconfined	≥ 0.4	3
Brittle Columns		1

The Turkish Draft Earthquake Code for seismic rehabilitation (TDEC 2005) permits structural components to exceed their DCR limits, provided that the number of such beams in a story is less than 20% of the total number and the contribution of such vertical members (columns, shear walls and struts) to story shear is less than 20%. A base shear capacity/demand ratio is defined for each building as BS20/BSD, where BS20 is the base shear capacity corresponding to the demand which causes vertical members that contribute less than 20% of the story shear in the critical story to exceed their DCR limit for collapse prevention, and BSD is the base shear demand. This ratio is calculated in both orthogonal directions and both senses, and the lowest value is employed for safety assessment. The distribution of BS20/BSD ratios for the 373 buildings is shown in Fig. 1. Four safety categories are defined accordingly, as indicated in Fig. 1. It is not surprising to observe that most buildings are at high risk, since they were initially screened out as having high risk. The assessment phase has indicated that 21 buildings can provide capacities at least 80 % of the code demand, which are categorized as low-risk. The highest ratio of low-risk buildings were in the 7-9 story level (24%). This leaves 352 buildings for which retrofit solutions are required.

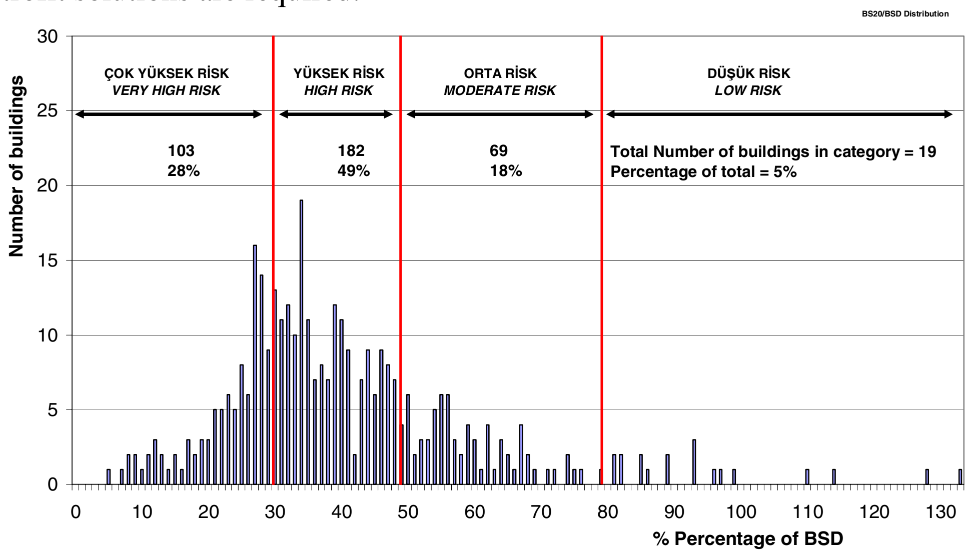


Figure 1. Building performance summary

Retrofit Solutions

A smaller set of 40 buildings is selected for detailed retrofit analysis and design. They are representative of the different types in the project population, and the results of their analyses have been extended to other similar buildings by judgment.

Selection of Representative Buildings

The buildings have been sorted first with respect to the following parameters, considering that they are the most influential characteristics in determining the retrofit solution alternatives.

- Number of floors above ground (i.e. effective number of floors),
- Base shear capacity/demand ratio (BS20/BSD),
- Building location, adjacencies (independent, end, middle or corner building),
- Presence of heavy overhangs (yes or no),
- Plan layout (square, rectangular, L, T or H shaped, other),
- Mean concrete strength obtained from core sample tests (6 to 22 MPa for individual buildings, with an average of 11 MPa for the entire 373 buildings).

The objective of the selection process is to identify 40 buildings that represent the distribution of the above parameters in the 373 buildings effectively. This selection process resulted in one 3 floor building, two 8 floor buildings, eighteen 4-5 floor buildings and nineteen 6-7 floor buildings. The distribution of BS20/BSD ratios for the sample buildings in each floor group were similar to the ratios given in Fig. 1. Further, the presence of heavy overhangs, plan layout and mean concrete strength in the sample of 40 buildings represented the entire 373 buildings fairly well.

Evaluation of the Performances of Retrofitted Buildings

3-D nonlinear static (pushover) analysis is employed for calculating the performances of sample buildings in their existing and retrofitted states by using the program SAP2000. Unconfined concrete model is employed for the existing concrete members and confined concrete model for the new members in determining the moment-curvature relationships for columns, shear walls and beams. Mean strength values are used for the existing materials, and design strength values for the new materials. For columns, beams and shear walls, plastic hinges are assigned to each end of a member where the plastic hinge lengths are taken as half the member depth. A typical moment-rotation relationship for a frame member is shown in Fig. 2. A residual moment of 20% of the ultimate moment is modeled. The nominal residual moment is a modeling contrivance to allow the analysis to continue once a member is ineffective in the lateral load resisting system. However a check was made to ensure that when a particular member reaches the rotation associated with onset of the residual moment (failure), the removal of the particular member does not lead to failure of the building. The ultimate rotation indicated in Fig. 1 is calculated by using the ultimate material strains, and an axial force level for columns derived from an elastic analysis in each direction for axial loads due to G+0.3Q+(E/R) loading. R is taken as 4.

Stress-strain relationship for unreinforced masonry infill struts is based on the strut models proposed by FEMA 356 (FEMA 2000) shown in Fig. 3, using E_{strut} = 2000 MPa; f_{strut} = 2 MPa; τ_{strut}= 0.12 MPa, ε_{fs} = .01 and ε_{crs} = ε_{so}. A residual stress of 20% of the ultimate is used. However a check is made to ensure that when a particular strut reaches the strain associated with onset of the residual stress (failure) the removal of the particular strut does not lead to failure of the building.

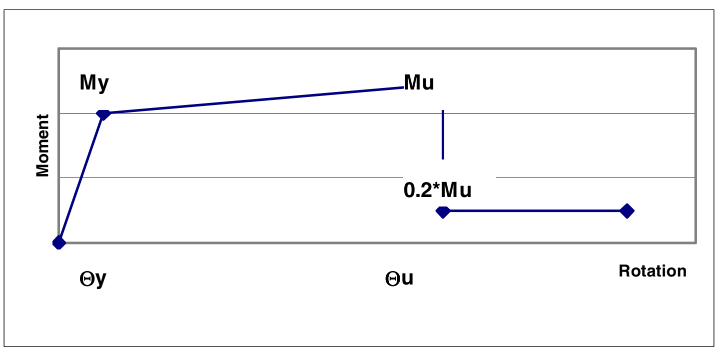


Figure 2. Typical moment-rotation relation for column and beam plastic hinges

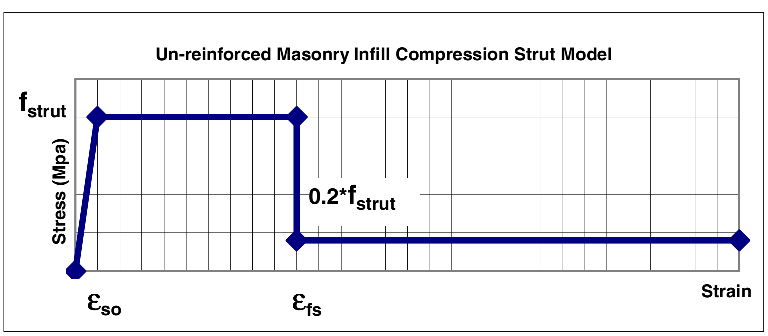


Figure 3. Compression strut model for unreinforced masonry infill walls

Accidental eccentricity effect in the structure is not included in the analysis procedure. For every building, analyses are completed with the load applied separately at 0, 90, 180 and 270 degrees. The vertical distribution of lateral forces is the distribution of lateral modal storey forces for the first mode found from an elastic modal analysis of the building, for the direction under consideration. Structural member connections are assumed to be infinitely rigid in 3-D mathematical models. Foundation supports to columns and shear walls are modeled as fully fixed.

Assessment of Capacity

The capacity curve is expressed graphically as the plot of spectral displacement versus spectral acceleration determined from the pushover analysis. It does not extend past the point where either the ultimate rotation limits of frame members, ultimate axial strains of struts, or the shear limits are exceeded, unless the affected member is not essential for the stability or integrity of the building. The Life Safety Capacity is taken as the point on the pushover curve corresponding to 75% of the spectral displacement at which the pushover analysis terminates or 75% of the displacement at which shear failure of a critical element occurs. For the retrofitting phase, the target performance level is “life safety”.

The shear capacity of the existing columns and shear walls are checked to ensure that shear does not limit the performance of the building. The new members are to be designed and checked according to the shear at performance point. If the existing member shear, as assessed from the pushover analysis, is greater than the maximum available shear capacity of the member, the capacity of the building as determined from the pushover analysis is reduced until the critical member shear is just reached.

Assessment of Demand

The demand response acceleration spectrum is derived from the 5% damped elastic (R=1) site specific spectrum (mean+1sd) for the site soil conditions appropriate for the building under consideration, scaled by $K_\xi \cdot S_p$, where S_p depends on the available ductility as follows.

$$\mu \geq 2 : S_p = 0.75; \quad \mu < 2 : S_p = 1.0 \tag{1}$$

An available ductility of 2 may be assumed unless the structure is expected to be particularly brittle. $\mu \geq 2$ is the typical situation, where $\mu = S_{d,\ ultimate} / S_{d,\ yield}$. The 5% damped response values for the scenario earthquake are modified for other damping levels by multiplying by K_ξ where $K_\xi = [7/(2+\xi)]^{1/2}$ and ξ is the equivalent viscous damping level (inherent damping plus hysteretic damping). The following equivalent viscous damping levels in Table 2 are assumed.

Table 2. Damping reduction factors proposed for different structural systems

Building type	Equivalent viscous damping	K_ξ
Non ductile RC frames	5%	1.00
Non ductile RC frames with infill masonry walls	10%	0.76
Buildings retrofitted with FRP to masonry walls	15%	0.64
Buildings retrofitted with steel “K” or “X” bracing	20%	0.56
Buildings retrofitted with RC-PP to masonry walls	20%	0.56

Buildings retrofitted with ductile concrete members (shear walls or coupled shear walls)	20%	0.56

The demand curves are found by plotting the demand spectral acceleration spectral ordinates against the demand spectral displacements, for the same ω_n or T_n.

Verification of Performance

The capacity and the demand curves in ADRS format are plotted on the same graph. On the capacity curve, the "Collapse Point" is taken to be the lower of:

a) The end point of the pushover curve from the analysis,
b) The point at which the shear check indicates failure of a critical member.

By using the 75% of the spectral displacement of the Collapse Point, the Life Safety (LS) performance level is defined. The required performance is achieved if the LS point on the capacity curve is at or beyond (to the right of) the demand spectrum.

Developing Retrofit Solutions

Two basic retrofit solution alternatives have been developed for each building:

- ***"Interior Solution"*** consisting mainly of cast in place shear walls integrated concentrically into an existing concrete frame by proper anchorage.
- ***"Exterior Solution"*** consisting of perimeter coupled shear walls attached and anchored to the slabs from outside at each floor level.

An interior solution has the advantage of design flexibility; however it needs partial or full evacuation of the building during construction. An exterior solution on the other hand does not require evacuation, but there are technical difficulties in anchorages of the coupled shear walls with the existing slabs, and in preventing uplift of the coupled wall segments in tension especially when the coupled shear walls are attached to the overhanging portions of the slabs. In this case they do not take the advantage of the axial forces in columns which remain along an inside frame.

The two basic retrofit solutions have been supported by secondary retrofitting elements, which are obtained by strengthening the existing unreinforced masonry infill walls. One method is to apply diagonal FRP sheets, and another method is to cover a wall face by thin reinforced concrete precast panels (RC-PP). The stress-strain models employed for the FRP and RC-PP strengthened wall struts are shown in Figs. 4 and 5, which are calibrated by using laboratory experiments (Binici 2005, Tankut 2005). An important advantage of masonry wall strengthening as indicated is that their field implementation cause minimum disturbance to the occupants.

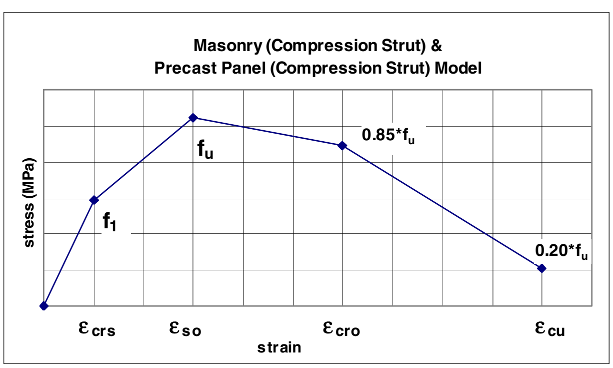


Figure 4. Compression strut model for masonry infill walls strengthened with RC precast panels. (ε_{crs}=0.001, ε_{cro}=0.003, ε_{cu}=0.01)

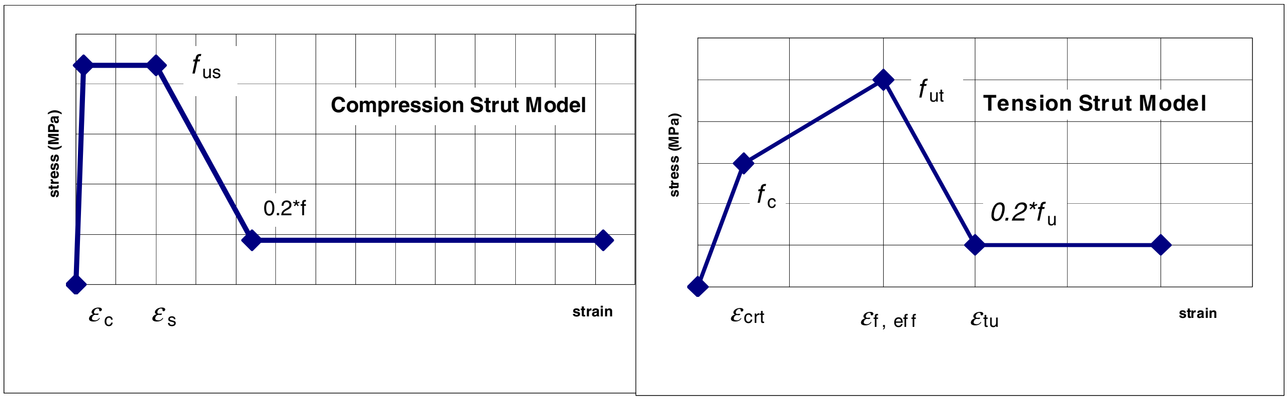


Figure 5. Compression and tension strut models for masonry infill walls strengthened with FRP

An important issue in the Bakirkoy Project was the low concrete strength. A practical criterion has been adopted in retrofit design, and it is decided that if the axial stress in a column under gravity loads exceed 60% of its measured concrete strength, then that column has to be jacketed regardless of the level of seismic forces.

Case Study

Retrofit solutions are presented for a six story independent building with overhangs along all four sides. Its mean concrete strength was 10 MPa, and it was in the high risk category. The picture of the building and plan of a typical story above basement is shown in Figure 6.

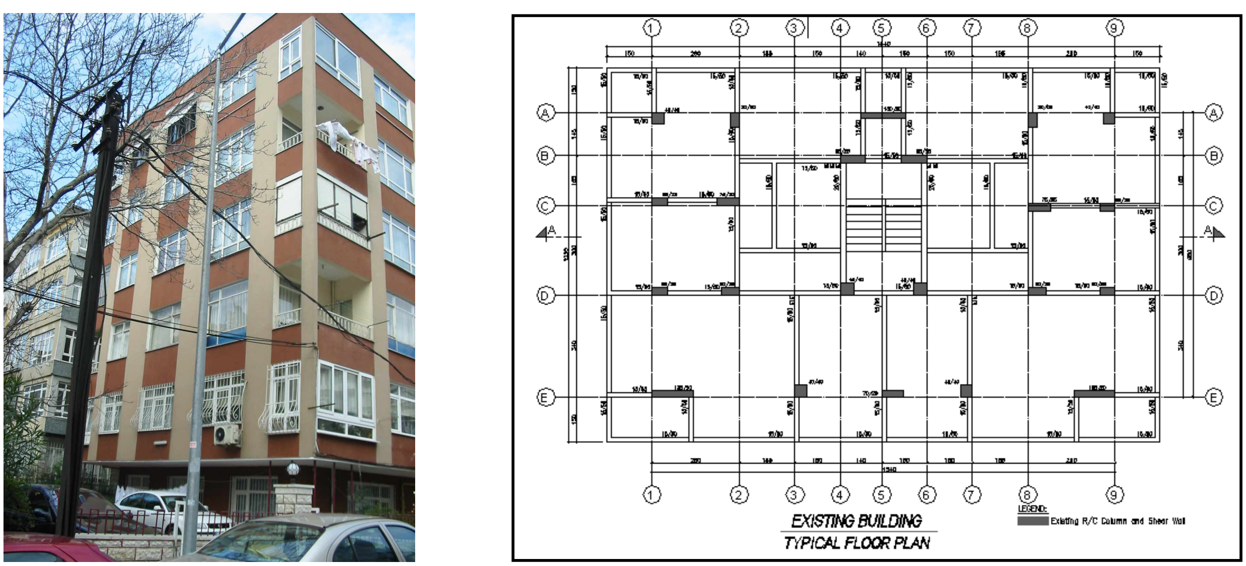
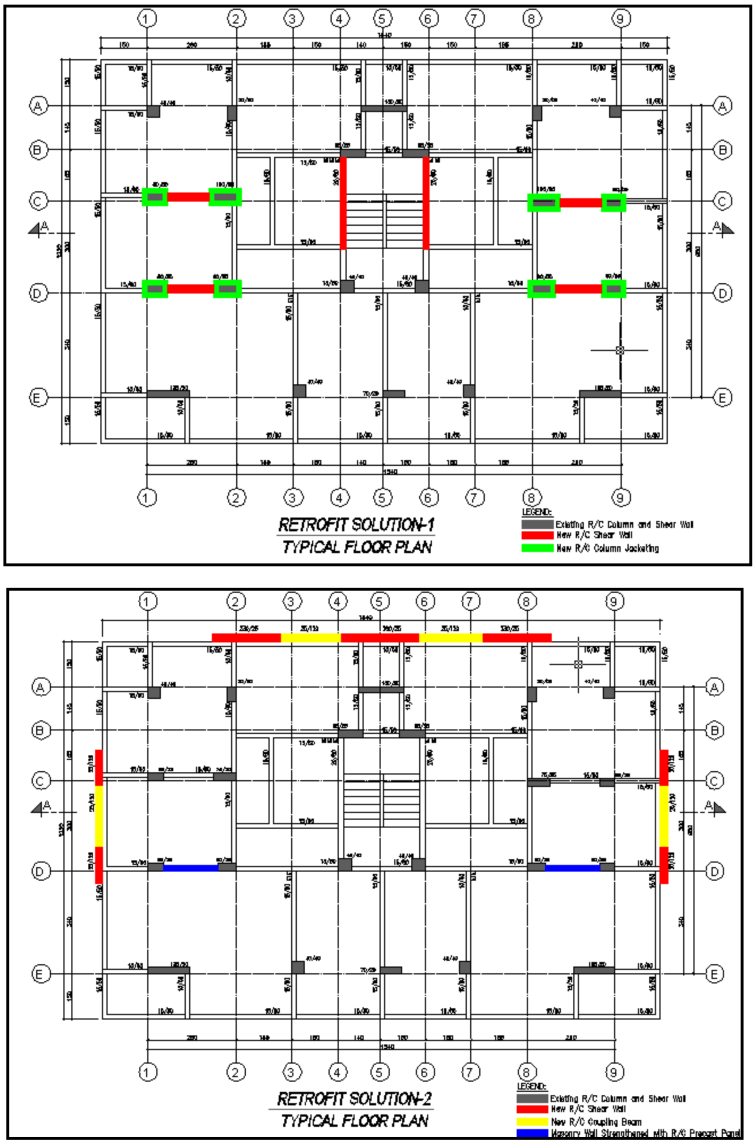


Figure 6. Side view and typical floor plan of the case study building in Bakirkoy

The interior and exterior retrofit solution alternatives are shown in Fig. 7. Two pairs of shear walls are employed in the interior solution in each direction, whereas the existing columns confining the new shear walls in the horizontal direction required jacketing in all floors. Three coupled shear walls are employed in the exterior solution, which are supported with secondary



(a)

(b)

Figure 7. (a) Interior and (b) exterior retrofit solutions for the case study building

RC-PP strengthened infill walls in the horizontal direction. It was not possible to attach a coupled shear wall on the front face of the building. The demand and capacity curves for the existing building and the two retrofit alternatives are shown in Fig. 8 in the ADRS format. The capacity supplied is sufficient to satisfy the demand in the life safety performance level although the displacement capacity of the exterior solution is less than that of the existing system in the X-direction. An acceptability criteria imposed on global displacements requires that the displacement demand for a retrofit solution at the performance point should not exceed the displacement capacity of the existing system. This is necessary to ensure that the existing members cope with the displacements of the retrofitted system.

The initial cost of retrofitting with respect to the replacement cost is calculated as 19% for the exterior solution and 39% for the interior solution. Furthermore the exterior solution has additional benefits when the life cycle costs are compared, due to the negligible downtime during construction. However the differences are less when column jacketing is considerable.

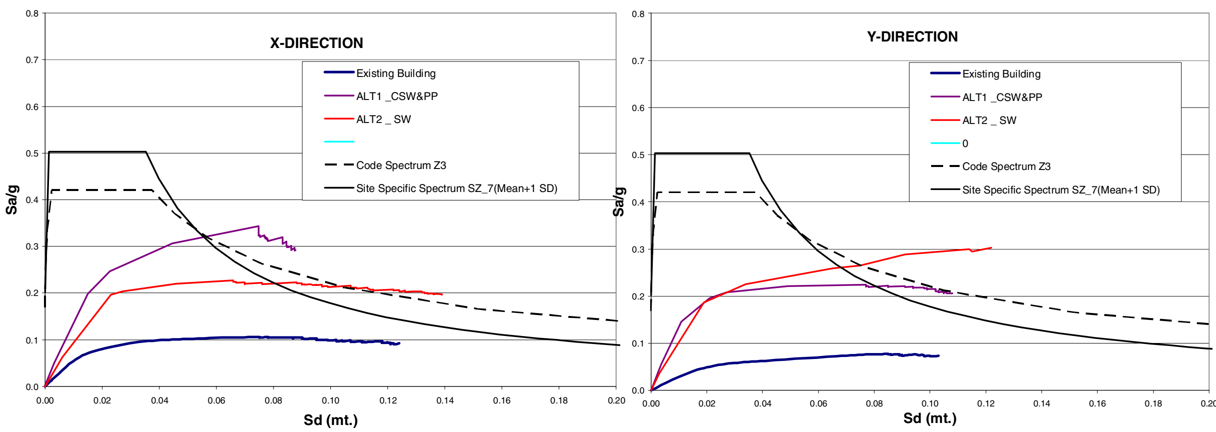


Figure 8. Demand and capacity curves of the case study building in ADRS format

Conclusions

The presented study reveals that feasible retrofit solutions can be developed for the high-risk building stock in Istanbul. Exterior retrofit solutions with perimeter coupled shear walls prevail in feasibility, with less cost and downtime. However foundation works in dense urban environment remain as a technical difficulty.

Acknowledgments

Bakirkoy Retrofit Feasibility Study, funded by the Government of Turkey through a World Bank loan is part of the Marmara Earthquake Emergency Reconstruction project. It was undertaken by the BECA/PROTA JV Consortium.

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