The 14th World Conference on Earthquake Engineering
October 12-17, 2008, Beijing, China

SEISMIC ASSESSMENT and REHABILITATION of A HISTORICAL UNREINFORCED MASONRY (URM) BUILDING IN ISTANBUL

Ufuk HANCILAR, Eser DURUKAL and Mustafa ERDIK

Bogazici University, Kandilli Observatory and Earthquake Research Institute, Department of Earthquake Engineering, Cengelkoy 34684 Istanbul, TURKEY
hancilar@boun.edu.tr, durukal@boun.edu.tr, erdik@boun.edu.tr

ABSTRACT:

The earthquake vulnerability of a historical building in Istanbul, which is a four-storey unreinforced clay brick masonry (URM) structure built in 1869, is evaluated and a conceptual design solution for seismic rehabilitation/strengthening of the building is proposed. The local strength characteristics of the brick walls are assessed based on the Schmidt hammer testing. Dynamic properties of the building (fundamental vibration periods) are checked with ambient vibration tests. The building is modeled and analyzed as a three-dimensional assembly of finite elements using *SAP2000 v11* (Static and Dynamic Finite Element Analysis of Structures) software package. The dynamic analysis procedure of FEMA-356 (Pre-standard and Commentary for the Seismic Rehabilitation of Buildings, ASCE, 2000) is followed for the detailed seismic assessment of the building. In order to improve earthquake resistance of the building, reinforced cement jacketing of the main load-carrying walls and application of fiber reinforced polymer (FRP) bands to the secondary walls are proposed.

KEYWORDS: Unreinforced Masonry (URM), historical building, seismic assessment, strengthening.

1. INTRODUCTION

Structures of brick masonry building are defined by vertical and horizontal elements, respectively by walls and floors. Floors should be rigid in their plane to distribute the seismic load among the vertical wall elements in proportion to their stiffness. Such floors are referred to as horizontal diaphragms. However diaphragms alone will be inadequate unless good connection between them and the supporting walls exists. In the event of an earthquake, apart from the existing gravity loads, horizontal loads are imposed on buildings. For URM buildings the walls, as load carrying members, provide the capacity to resist the demand created by these gravity and earthquake loads. Surveys of earthquake damaged URM buildings show that well tied buildings with well defined, continuous load path to the foundations will have a much better performance in earthquakes. Well defined continuous load path can be achieved with regular structural layout with symmetry and uniformity both in plan and elevation. The creation of tensile and shearing stresses in walls of masonry buildings exposed to strong earthquakes is the primary cause of damage suffered by URM buildings. For such buildings with inadequate capacity, exposed to severe and prolonged earthquake ground motions the cracks become wider and the masonry units become loose causing partial collapse and gaps in walls occur due to falling of loose masonry units. Eventually walls get separated at corners and fall outwards leading to either partial or full collapse.

The objective of this study is to determine the earthquake performance of a historical URM building and to propose strengthening techniques in order to improve its earthquake resistance. The building is a four-storey unreinforced clay brick masonry (URM) structure built in 1869. Its general dimensions are 19.9 x 23.7m in plan and its height is 15.5m. It arises on a semi-basement floor and three normal stories (Figure 1). The building is currently being used for office and residential purposes. It survived the 1894, M_s7.0 Istanbul earthquake, during which widespread damage to URM buildings took place in the city (Ambraseys and Finkel, 1991). No information could be reached whether and in what way the building was affected by this earthquake. After elaborating a three-dimensional finite element model of the building, dynamic analyses following the FEMA-356 procedure are conducted to assess the performance of lateral load carrying system. The walls that need to be strengthened are determined based on the demand-to-capacity ratios. For the rehabilitation of the walls, reinforced cement jacketing of main load-carrying walls and application FRP bands to the secondary walls are proposed.

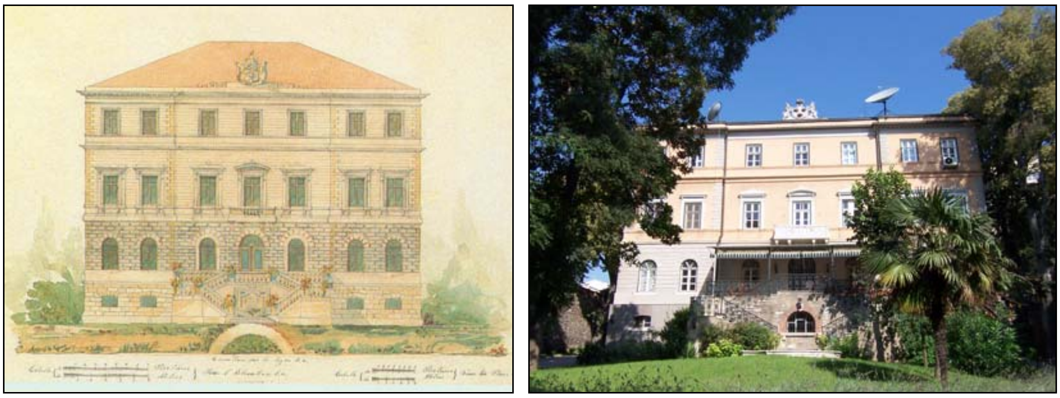

Figure 1. The original drawing of the building after Pulgher (1869) and a recent photo (right)

2. EARTHQUAKE HAZARD ASSESSMENT

Deterministic seismic hazard assessment was conducted to determine the spatial distribution of the design basis earthquake ground motion for the site that would result from a deterministic (scenario) earthquake. The design basis ground motion was assessed at the so-called free-field outcrop of the reference soil media, which is be the engineering bedrock of NEHRP B/C boundary soil class with an average shear wave propagation velocity of 760 m/s. This reference soil media correspond to the upper layers of the so-called Trakya Formation consisting of claystone, sandstone and siltstone. This zone can essentially be assumed to be soft rock with an average shear wave propagation velocity of 750m/s (Site Class B/C boundary in NEHRP, 1997). An M_w=7.5 (similar to 1999 Kocaeli earthquake in magnitude and in total rupture length) was selected as the "Maximum Credible Earthquake- MCE" scenario event. Using the resulting median values of spectral accelerations at 0.2s and 1s at the site of the building, the 5%-damped elastic response spectrum can be plotted (Figure 2).

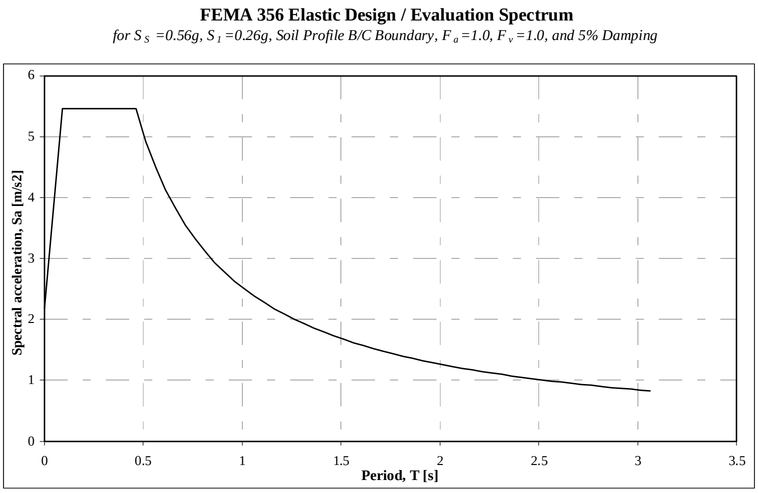

Figure 2. 5%-damped elastic response spectrum

3. EXPERIMENTAL STUDIES

Two series of experiments were carried out to determine dynamic properties of the building and the strength characteristics of the construction materials: Ambient vibration tests and Schmidt hammer testing.

3.1. Ambient Vibration Tests

Ambient vibration tests are carried out to determine the dynamic properties of structural systems. The tests make use of the natural noise affecting a structure arising from sources such as human activity, wind, traffic and so on and measure the structural response to these sources. The assumption is that a structure will respond in a similar manner to these low-amplitude vibrations as it would respond during an earthquake and thus the dynamic structural properties deduced from ambient vibration records are similar to those found from earthquake recordings.

Ambient vibration tests were carried out in the building with four Kinematrics SS-1 1-D seismometers. The seismometers were distributed among the four floor of the building, so that there was one seismometer in each floor along the same vertical axis. The records were obtained in two principal axes of the building with the aim of determining the dynamic properties in two directions. The experimental setup consisted of four seismometers, one signal conditioner, A/D converter and the LabView digital signal processing software. Data correction involved instrument correction, baseline correction and filtering between 1 and 10 Hz. The records obtained in Volts are proportional to velocity. The fundamental structural vibration frequencies are identified from the power spectral densities (in Vsec). Smoothed power spectral densities are presented in Figure 3. On the basis of ambient vibration tests and subsequent data analysis following fundamental frequencies of vibration could be identified:

4.3 Hz - direction X, translation
4.2 Hz - direction Y, translation

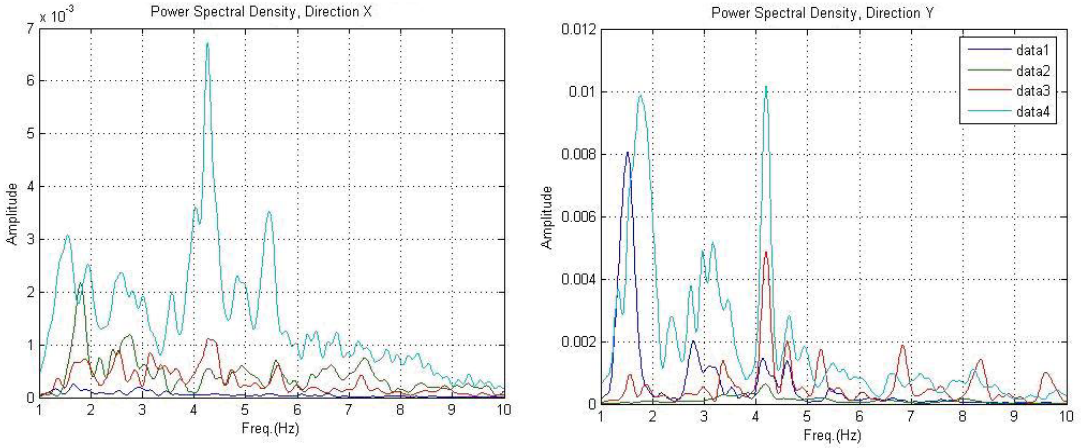

Figure 3. Smoothed power spectral density functions: X-dir. (left) and Y-dir.(right). Shown in blue are the recordings obtained at the basement, in green at the ground, in red at the 1st and in light blue at the 2nd floors.

3.2. Material Testing

The purpose of Schmidt hammer tests is to assessment of local strength of brick walls. Compressive strength and unit weight of the material can be assessed as a result of analysis. The experiments are applied to sound bricks. Schmidt hammer testing is generally used for the determination of strength properties of concrete. Its usage in masonry structures should be interpreted with caution due to the composite nature of masonry and due to limited availability of calibration data from tests carried out on masonry structures.

Schmidt hammer tests were carried out at seven locations in the basement of the building (Figure 4) and the average rebound number is found as 35.4. On the basis of the values given in the literature following compressive strengths can be determined:

Average: 3.3 MPa, lower bound: 2.8MPa, upper bound 22 MPa, as per NAVFAC (1992).

Lower bound can be estimated as < 2.9MPa, as per Tassios and Mamillan (1994).

It should be noted that these measurements relate to particular points of single bricks and are naturally associated with large variations. The tendency in the practice is to consider the lower strength values of measurements to provide an adequate margin of safety.

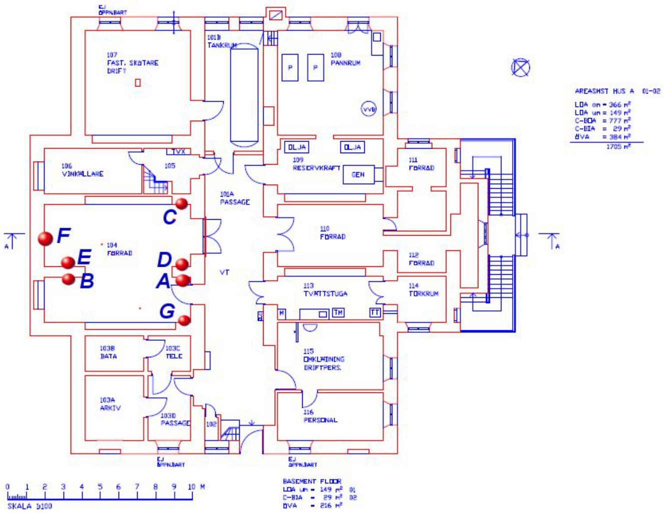

Figure 4. Locations of Schmidt hammer testing in the basement of the building. Letter A to G indicate the locations of Schmidt Hammer Testing.

4. SEISMIC PERFORMANCE EVALUATION

For the earthquake performance assessment of the building the following procedure is utilized:

1. The performance assessment earthquake is set to be equal to the median site-specific earthquake ground motion that would result from the moment magnitude M_w=7.5 earthquake that would take place on the Main Marmara Fault, passing about 18 km south of the site of the URM building.

2. For the assessment of the structural capacities of the load carrying elements the expected strengths (Q_{CE}, FEMA-356 terminology) were used. The expected vertical compressive strength of the load carrying element is given by (FEMA-356, 7.4.2.2.3):

$$Q_{CE} = 0.7\, f_m\, A_n \tag{4.1}$$

where A_n is net cross-sectional area of the element and f_m is the compressive strength of the masonry. EC-8 and FEMA-356 foresees a minimum value of f_m = 2MPa. Turkish Earthquake Resistant Design Code (TERDC-2007) stipulates the following minimum expression for Q_{CE}.

$$Q_{CE} = 1.0\, A_n \tag{4.2}$$

As such, 1MPa replaces the 0.7 f_m in the FEMA-356 equation.

The expected lateral shear strength of the load carrying element is given by (FEMA-356, 7.3.2.6 and 7.4.2.2.1):

$$Q_{CE} = (0.08 + 0.5\,\sigma)\, A_n \tag{4.3}$$

where σ is the compressive stress on the load carrying element. Turkish Earthquake Resistant Design Code

(TERDC-2007) stipulates the following minimum expression for Q_{CE}.

$$Q_{CE} = (0.15 + 0.5\,\sigma)\,A_n \qquad \textbf{(4.4)}$$

The first term in these equations relate to the tensile strength of the masonry.

3. To obtain the earthquake performance of the load carrying component or element as per the Linear Dynamic Procedure of FEMA-356, their structural capacities (Q_{CE}), modified by the so-called "knowledge factor" (κ), are compared with their earthquake demands (Q_{UD}).

$$m\,\kappa\,Q_{CE} \geq Q_{UD} \qquad \textbf{(4.5)}$$

where Q_{CE} is the expected strength of the component or element, κ is the knowledge factor and was assumed to be 0.75 for the building under consideration. m is the component demand to capacity ratio provided in Table 7.3 of FEMA-356 for different performance levels as: $m=1$ for "Immediate Occupancy", $m=3$ for "Life Safety" and $m=4$ for "Collapse Prevention". Q_{UD} is the force calculated due to the gravity and earthquake loads:

$$Q_{UD} = Q_G \pm Q_E \qquad \textbf{(4.6)}$$

$$Q_G = 1.1\,(Q_D + 0.25\,Q_L) \text{ and/or } Q_G = 0.9\,Q_D \qquad \textbf{(4.7)}$$

where Q_D is dead load and Q_L is effective live load.

4.1. Structural Response Analysis

The building was modeled, analyzed and evaluated as a three-dimensional assembly of finite elements. The linear dynamic analysis procedure of FEMA 356- Prestandard and Commentary for the Seismic Rehabilitation of Buildings (ASCE, 2000) was followed for the detailed seismic assessment of the building. Similar procedures are also sanctioned by Euro Code and the relevant Turkish Codes.

Fundamental vibration periods of the building resulting from the free vibration analysis are:
T_x=0.202 sec.
T_y=0.193 sec.

The similar values of the fundamental vibration periods were obtained from the ambient vibration tests as follow
T_x=1 / 4.3 = 0.232 sec.
T_y=1 / 4.2 = 0.238 sec.

In order to assess dynamic response of the building modal spectral analysis was performed using linearly-elastic response spectrum given in Figure 2. The storey shear forces acting through each horizontal direction resulting from the dynamic analysis is given Table 1.

Table 1. Analysis Results Summary

Storey	Storey Height (m)	Seismic Weight (kN)	Storey Shear Forces in X Direction (kN)	Storey Shear Forces in Y Direction (kN)
2nd	3.5	4298	3203	3080
1st	4.5	6042	6465	6444
Ground	4.5	6786	8483	8465
Basement	3	7180	9186	8986
Total	15.5	24307		

The governing behavioral mode of the walls is bed-joint sliding shear that limits their expected lateral strength. The walls are assessed based on the demand-to-capacity ratios for three different performance levels, Immediate Occupancy, Life Safety and Collapse Prevention. The resulting shear stresses from the dynamic analysis are shown only on the walls that do not satisfy the Life Safety performance criteria in Figure 5. The shear stress contours are ranging between 0 and 1 MPa.

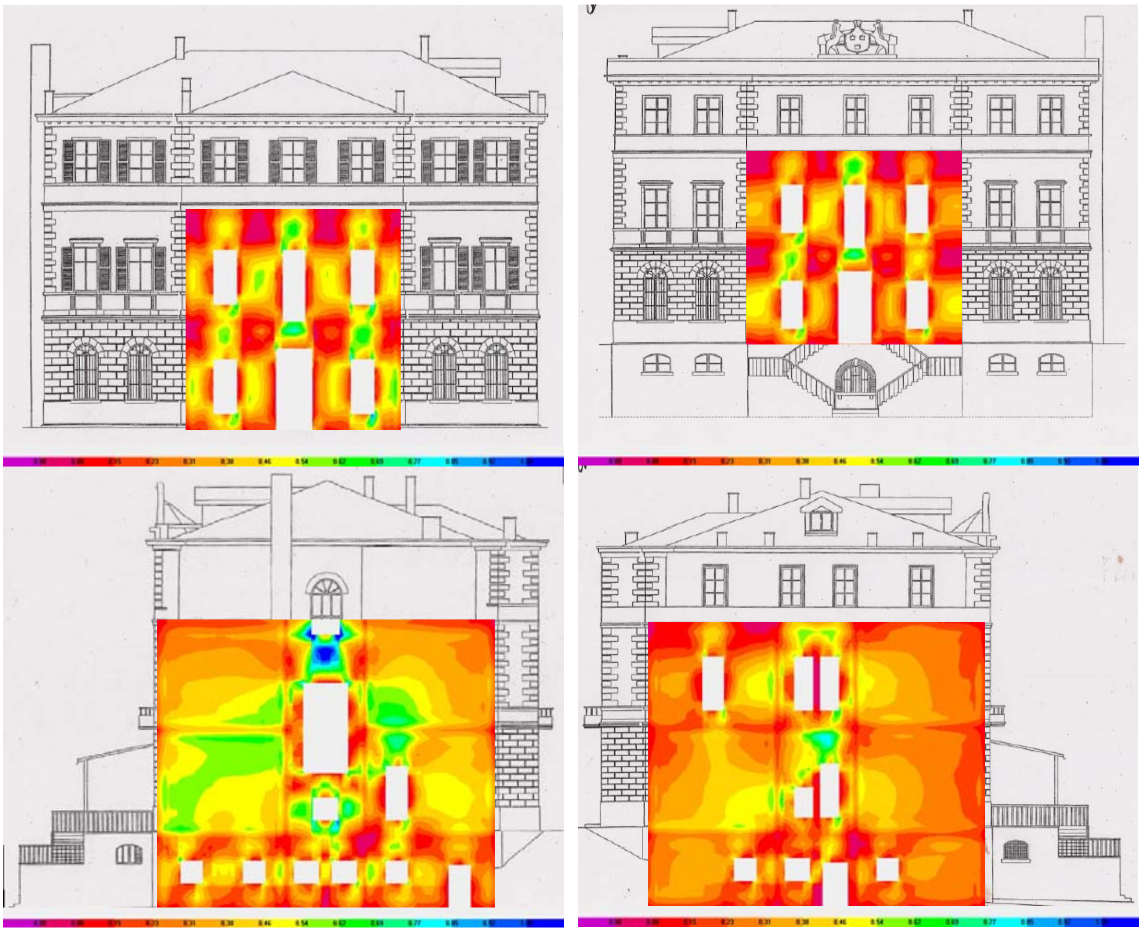

Figure 5. The critical walls with the shear stress contours to be strengthened on the facades of the building

5. REHABILITATION / STRENGTHENING OF INDIVIDUAL WALLS

The walls to be retrofitted are shown on the floor plans in Figure 6. Walls marked with red color will be strengthened by reinforced cement jacketing and use of fiber reinforced polymer (FRP) sheets/bands is suggested for strengthening of the walls marked with yellow color.

The reinforced cement jacketing may be applied either onto both faces of the walls or only onto one face. Required reinforcement areas for the walls to be retrofitted by reinforced cement jacketing are given in Table 2. The average required reinforcement amount can be provided by S500bs quality re-bars of 10Φ / 10 cm onto one face of the wall or 6Φ / 10 cm onto both faces of the wall. Required amount of FRP bands / sheets for strengthening of the walls marked with yellow color in Figure 6 will depend on the material characteristics of the selected brand.

Table 2. Required reinforcement areas for the walls to be retrofitted by reinforced cement jacketing

WALL	length (m)	thickness (m)	section area, A_d (m^2)	Shear Stress (MPa)	ρ_{sh} (%)	reinforcement area (cm^2) per m^2 of the wall
W28	6.50	0.75	4.88	0.57	0.06	4.77
W29	3.95	0.75	2.96	0.51	0.05	3.94
W37	6.50	0.75	4.88	0.43	0.04	2.77
W84	6.50	0.75	4.88	0.63	0.08	5.66
W88	3.95	0.75	2.96	0.67	0.08	6.31
W89	6.50	0.75	4.88	0.64	0.08	5.80
W93	6.43	0.55	3.54	0.59	0.07	3.69
W94	3.99	0.55	2.19	0.77	0.10	5.68
W95	6.48	0.55	3.56	0.59	0.07	3.77
W33	6.08	0.45	2.74	0.82	0.11	5.14
W39	6.18	0.45	2.78	1.17	0.18	8.28
W96	6.43	0.55	3.54	0.51	0.05	2.90
W97	3.99	0.55	2.19	0.89	0.13	7.01
W98	6.48	0.55	3.56	0.74	0.10	5.35
W41	10.50	0.55	5.78	0.95	0.14	7.75
W91	10.70	0.55	5.89	0.73	0.10	5.31
W73	6.43	0.45	2.89	0.64	0.08	3.49
W74	3.99	0.45	1.80	0.75	0.10	4.50
W75	6.40	0.45	2.88	0.62	0.07	3.33
W76	6.43	0.45	2.89	0.58	0.07	2.95
W77	3.99	0.45	1.80	1.16	0.18	8.17
W78	6.40	0.45	2.88	0.68	0.09	3.90
W48	10.30	0.45	4.64	0.81	0.11	5.04
W58	10.70	0.45	4.82	0.70	0.09	4.07

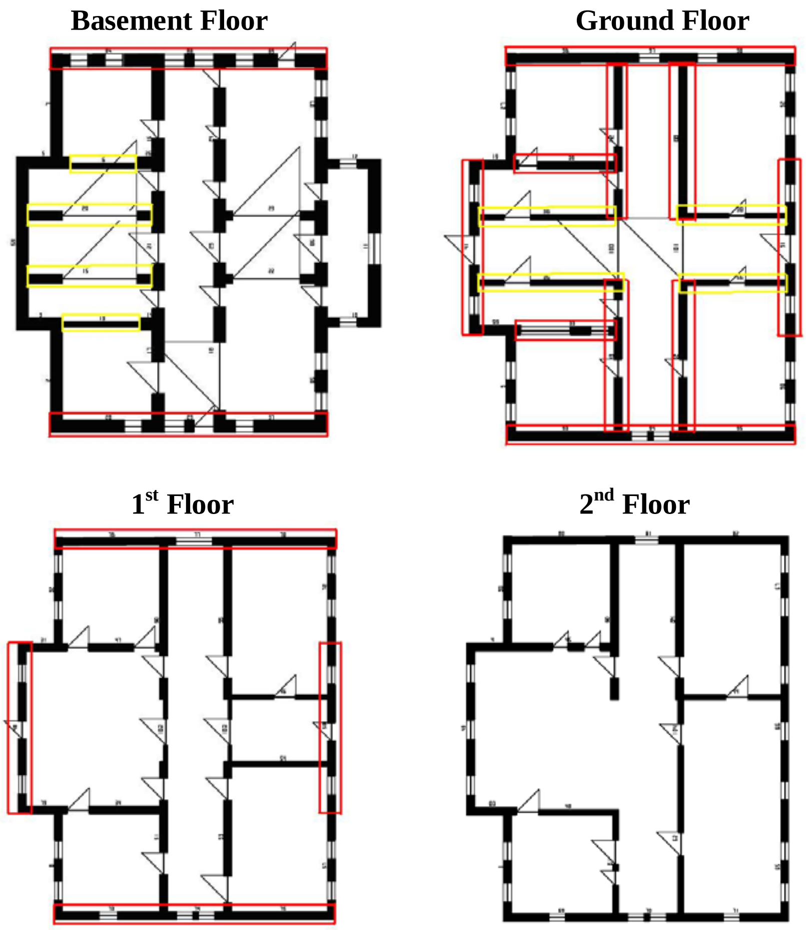

Figure 6. The floor plans and the walls to be retrofitted

6. CONCLUSIONS

The earthquake performance evaluation of a historical URM building is carried out and strengthening techniques are proposed to improve its earthquake resistance. In order to obtain dynamic response of the building, 5%-damped elastic response spectrum that results from the deterministic seismic hazard assessment considering the maximum considerable earthquake-MCE scenario event at the site of building is utilized. Similar values of fundamental free vibration periods are found from the ambient vibration tests and computer analysis. The strength characteristics of the unreinforced masonry (URM) walls are determined by Schmidt hammer testing. The seismic performance of the building is assessed based on the demand-to-capacity ratios of the individual walls. Reinforced cement jacketing for the main load-carrying walls and application of FRP sheets/bands for secondary walls are proposed.

REFERENCES

ACI 530-02/ASCE 5-02/TMS 402-02, (2002), Building Code Requirements for Masonry Structures, Masonry Standards Joint Committee, USA.

American Society of Civil Engineers, FEMA 356–Prestandard and Commentary for the Seismic Rehabilitation of Buildings, 2000.

Ambraseys, N. N. and C. F. Finkel (1991). Long-term seismicity of Istanbul and of the Marmara Sea region. *Terra Nova* **3,** 527-539.

Building Seismic Safety Council, FEMA- 274, NEHRP commentary on the guidelines for the seismic rehabilitation of buildings, 1997.
AEE.

Erdik, M. M.Demircioğlu, K.Şeşetyan, E.Durukal ve B.Siyahi (2004). Assessment of probabilistic earthquake hazard in the Marmara Region. *Soil Dynamic and Earthquake Engineering* **24**, 605–631.

LabView 8, National Instruments.

M. Tomazevic, M. EERI, M. Lutman, P. Weiss (1993). The seismic resistance of historical urban buildings and the interventions in their floor systems: an experimental study. *The Masonry Society Journal* **12: 1**.

M. Tomazevic, M. EERI, M. Lutman, P. Weiss (1996). Seismic upgrading of old brick masonry urban houses: Tying of walls with steel ties. *Earthquake Spectra*, **12: 3**.

Miha Tomazevic (1999). Earthquake Resistant Design of Masonry Buildings, Imperial College Press, London, U.K.

NAVFAC (1992), Masonry Structural Design for Buildings, Technical Manual No. 5-809-3, Navy Manual, NAVFAC DM-2.9, Air Force Manual No. 88-3, Chapter 3, Departments of The Army, the Navy and the Air Force.

NEHRP (1997), Recommended Provisions For Seismic Regulations For New Buildings and Other Structures, FEMA-303, Prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, DC.

SAP2000, Static and Dynamic Finite Element Analysis of Structures, Advanced v10.1.1, Computers and Structures Inc., California, USA.