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Seismic retrofitting of the historical masonry structures using numerical approach

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HIGHLIGHTS

- The laboratory tests and in situ analyses of samples taken from the building have been performed.
- The numerical analysis of the prepared building model using finite element software have been given.
- The restoration applications using laboratory, in situ and numerical analysis results have been presented.

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ABSTRACT

In historical buildings, because of the deterioration on structural members including slab and walls that complete their life cycle in time, due to environmental conditions; restoration applications have become necessary to increase the material durability level and to have adequate level of structural strength in order to resist dynamic effects such as earthquakes. This paper focuses on assessment of historical masonry structures from the point of seismic resistance. The entire process is illustrated using case study from a historical masonry structure. In this study conducted in this respect, a historical building is restored within the scope of laboratory studies and numerical analyses. The first stage of the study includes plaster analyses and mechanical tests conducted on the samples taken from the said building. In the second stage, i.e. numerical analysis, the building's existing 3D computer model was prepared and materials, members that are inadequate in terms of strength were determined. The third stage includes restoration applications by using laboratory and numerical analysis results. Within the scope of restoration applications, structural cracks on the walls were repaired using the injection method; volta slab (brick floor arches), exterior facade walls, interior walls and door/window gaps using different techniques were strengthened. In this study, it was aimed to increase material durability and structural strength by using conventional and modern techniques within the scope of laboratory tests and numerical approaches in recovering the historical buildings.

1. Introduction

The conservation, preserving and restoration of historical masonry structures belonging to the cultural heritage, strengthening their main structural members, are becoming a very important issue in Turkey. Therefore, the structures need restoration to survive their life as a result of aging and increasing load demand. Many historical structures have been restored in order to resist these effects [1]. Masonry constructions are typically complex structures and there is lack of knowledge and information concerning the behavior of their structural systems, particularly in what regards their seismic response. Typically, these structures are more massive than today's structures and usually carry their actions primarily in compression [2]. According to results of the work developed within the ICOMOS 2001 recommendations, a thorough understanding of the structural behavior and material characteristics is essential for any project related to the architectural heritage. It is recommended that the work of analysis and evaluation should be done with the cooperation of specialists from different disciplines, such as earthquake specialists, architects, engineers and art historians. In addition, it is considered necessary for these specialists to have common knowledge on the subject of conserving and upgrading or strengthening the historical buildings [3].

The historical building in which restoration was carried out is located in Istanbul University land in Fatih district, Süleymaniye

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parish, 579 building block, 1 plot and in the historical peninsula of Istanbul which is listed in the World Heritage List of UNESCO. This historical building is registered by Istanbul IV Cultural and Natural Heritage Preservation Board as a cultural heritage to be preserved with the preservation group I [4,5]. The historical building is named as Beyazit secondary school in German blue map dated 1911–1913 while it is named as Court of First Instance in Pervititch map dated 1935, indicating changes in the function of the building [6,7]. It is stipulated that the latest function of the building, before it was handed over to the University, is student dorm within the Ministry of National Defence, a period in which it saw the most significant changes.

Preparing the restoration project, historical photos and old maps were utilized in collecting data and documents on the building facade; therefore, information on structure form, roof form, storey height of the building, roof patio, storey count, relationship with the neighbouring buildings and location of the windows was obtained [6–8]. Views of the historical building before the restoration are presented in Fig. 1.

In order to improve the unfavourable condition of the said building in terms of safety, function and aesthetics and bring it in compliance with its historical identity, relief and restitution project based on old-dated maps and photos were prepared and a restoration project was drafted based on this data. The restoration project was found suitable and approved by the decree of Istanbul No. I Regional Board of Preserving Cultural Heritage [9].

A number of researches have been carried out to investigate the seismic resistant of historical masonry structures [2,10–28]. Asteris et al. present a methodology for earthquake resistant design or assessment of masonry structural systems [2]. Valluzzi et al. investigated that the structural rehabilitation of monumental area. In the paper, after a general presentation of the main properties and of the most relevant deterioration phenomena of the principal parts of the monumental area, the methodology that is being used for the structural diagnosis, for the implementation of guidelines for the future interventions and for the maintenance of the restored conditions are presented [15]. Aktas and Turer focused on seismic evaluation and strengthening of Nemrut monuments. The simulations showed vulnerability of cut-stone blocks separating from one another under seismic action, and simple strengthening solutions were proposed [20]. Mele et al. analysed a basilica-type church in order to assess its structural behavior and seismic vulnerability. For this purpose, an effective two-step procedure has been used, consisting of 3D static and dynamic linear analyses of the structural complex, and 2D nonlinear push-over analysis of the single macro-elements [24]. Bernardeschi et al. described the numerical techniques implemented in the finite-element code NOSA for structural analysis of masonry constructions [27]. Abruzzese et al. evaluated the risk of collapse of the Huzhu Pagoda, one of the oldest masonry pagodas built in the XI century. In the study, mechanical properties of the masonry material have been obtained by experimental tests on small specimen and the mechanical behavior of the structure has been evaluated via numerical models. The static analysis of this ancient pagoda constitute a prerequisite base for the evaluation of its structural behavior leading to a suitable maintenance program [10]. Barbieri et al. performed a structural analysis of a historic masonry building subject to significant static instabilities due to an overturning of the longitudinal facades related to ground settlements [19]. Mahini proposed a macro-modelling approach and the performance of a CFRP-retrofitted in historical building. The brick and adobe, prism samples of the building have been modelled by commercial code, which uses smeared-crack materials and eight-noded isoparametric, solid elements [14].

In this paper, restorations applications in a historical building within the scope of lab tests and numerical analyses were presented within the scope of laboratory tests, numerical analyses and a cross-disciplinary study of civil engineering and architecture. Conventional and modern techniques were based on intervention decisions as a result of both laboratory studies and numerical analyses on the building. This paper is aimed to become an exemplary study in retrofitting applications in historical buildings, especially in terms of increasing the seismic resistant.

Fig. 1. Facade views of the structure the pre-restoration (a–f).

2. Laboratory studies

Samples were taken from different points of the building during lab test stage of the study and analyses were carried out. As a result, axial compressive strength of the masonry walls, shear strength, bulk density, thick and thin plaster compounds were obtained. Also, mechanical characteristics of bricks and pointing fillings were obtained.

2.1. Plaster and mixture analyses

Based on result of exist plaster analyses, plaster mixture types to be used during the application were decided. Grout additive was added to the mixture in order to improve mechanical properties of the thick plaster mixture (first layer) while polypropylene fiber – used in lime-white cement mixtures and locally known as "kıtık" (plasterer's hair) – improving mechanical properties of the mortar mixture [9] was used in thin plaster mixture. In line with original plaster analyses, the proposed thick- and thin-plaster mixture (second layer) ratios are presented in Table 1.

Mixture ratios of the samples taken to repair the structural cracks in masonry walls were examined. The ratios of the mixture used in the restoration injection applications for structural strengthening and repair of the historical masonry walls, specially structural cracks, are given in Table 2.

Table 1
Thick- and thin-plaster analysis in proposed restoration plaster.

Material	Thick plaster mixture ratio	Thin plaster mixture ratio
Slaked lime	1 volume (water ratio ~50%)	1 volume (water ratio ~50%)
Hydraulic lime	1/3 volume	1/3 volume
Stream sand (washed)	1½ volume, 2 mm undersize	1½ volume, 0.5 mm undersize
Brick dust	1½ volume	1½ volume
Water	8/25 volume	8/25 volume
Polypropylene fiber (0.6 kg/m^3)	25 g (in 20 L mortar)	25 g (in 20 L mortar)
Grout additive	(added)	(none)

Table 2
The mixture ratios for structural injection applications.

Material	Mixture ratio
Hydraulic lime	1 volume
Slaked lime	1 volume (water ratio ~50%)
Brick dust	1 volume
Water	2 volume (5% acrylic resin emulsion)

2.2. Determining design parameters

2.2.1. In situ wall shear strength indexes

Shear tests based on the ASTM C 1531 standard [29] were performed on masonry walls and shear strength index values of brick-mortar mixture were obtained. Shear tests were performed in six different points in the building. Samples were taken representing the brick-mortar-brick system, compressive strength and bulk density tests were performed on such samples in the laboratory (Table 3).

2.2.2. Mechanical and physical characteristics of solid brick samples and pointing filling mortars

Samples from the solid brick blocks and pointing filling mortars were taken from the examined structure. Single axis compressive strength and bulk density tests were carried out on brick blocks. In order to determine the dead weight of the wall, density of filling mortars between the bricks were determined (Tables 4 and 5).

2.2.3. Wall elastic moduli

As a result of the tests performed on the building; in parallel to the data and experiences obtained from other masonry buildings having similar historical process, production conditions and materials, it was decided that the elastic modulus, considering the existing condition of the existing walls, would be reflected on the structural model with values ranging from 400 to 600 MPa [30,31].

3. Finite element model

Sectional and geometric parameters related to the existing load-carrying system were determined by means of the relief application on the building. The structure has 4-storey masonry load-carrying system including three regular storeys and one attic. It is the shape of irregular with the dimension of 14.76 m × 15.04 m. The building's housing space is around 222 m^2 while its usage area is 888 m^2. Floor system is volta slab and is the same in every storey. The said floor system originally consists of steel I profiles of 0.25 m height, solid bricks and clinker materials. The building within the scope of the study consists of inadequate and independent attachments with various usage purposes, different geometries. Storey heights are 3.75 m, 4.84 m, 4.48 m and 3.10 m for the ground-, first-, second-floors and the attic respectively while the tiling thickness varies between 0.24 m and 0.27 m. Also, brick masonry thickness is calculated 0.44–0.58 m on the ground floor, 0.41–0.56 m on the first floor, 0.43–0.47 m on the second floor and 0.23–0.40 m in the attic.

A three-dimensional, finite element analysis of the four storey masonry building under seismic loading is carried out. Wall shear

Table 3
Shear test results in situ.

Sample no	Storey	Brick dimensions			Mortar thicknesses		Ultimate load (kN)	Shear strength index (MPa)
		Width a (mm)	Length b (mm)	Height h (mm)	Up (mm)	Down (mm)		
1	Ground	120	125	70	10	25	15.8	0.41
2	Ground	130	60	75	20	10	8.0	0.41
3	Ground	120	110	65	15	20	18.4	0.55
4	First	110	120	60	25	15	20.8	0.62
5	First	120	120	65	25	10	31.9	0.87
6	Second	105	120	70	20	15	25.4	0.76
7	Second	130	140	75	20	20	25.6	0.55
8	Second	115	110	70	20	15	24.2	0.73
9	Attic	120	115	70	10	10	12.4	0.35
Average shear strength, M, (MPa)								*0.58*
Standard deviation, σ, (MPa)								*0.18*
$M - \sigma$								*0.40*

Table 4
Compressive strength values of brick block samples.

Sample no	Storey	Sample dimensions (mm)			Area (mm^2)	Ultimate load (kN)	Compressive strength (MPa)
		Width, *a*	Length, *b*	Height, *h*			
1	Ground	98	82	71	8036	84.4	10.5
2	First	85	88	73	7480	90.5	12.1
3	Second	89	78	73	6942	77.8	11.2
Average							*11.3*

Table 5
Density of brick block samples with pointing filling mortar.

Sample no	Density (kN/m^3)
1	172.0
2	171.0
3	170.8
Average	171.3

strength to be used in numerical analyses was determined individually for each of the storeys, as a result of the material determination studies; linear elastic analyses identified their performances based on the existing building's load-carrying system data. All analyses were carried out by using SAP2000, a general purpose finite elements software, and according to calculation principles defined as per the relevant country's local specifications and standards [32,33,35]. The said structure, in terms of evaluating and reinforcement of the existing buildings, was analysed according to four different building performance levels for the existing or strengthened buildings, in addition to various level earthquake definition. These performance levels were defined as; i. Operational level, ii. Immediate occupancy level, iii. Life safety level, iv. Collapse prevention level or near collapse level (Fig. 2).

Building performance can be described qualitatively in terms of the: i. Safety afforded building occupants, during and after an earthquake. ii. Cost and feasibility of restoring the building to pre-earthquake conditions. iii. Length of time the building is removed from service to conduct repairs. iv. Economical, architectural, or historic impacts on the community at large. These performance characteristics will be directly related to the extent of damage sustained by the building during a damaging earthquake.

Building performance levels typically comprise a structural performance level that describes the limiting damage state of the structural systems, plus a non-structural performance level that describes the limiting damage state of the non-structural systems and components.

Dead and live loads used in the analyses are presented in Table 6.

Table 6
Dead and live load values [35].

Structural or non-structural member	Dead load (kN/m^2)	Live load (kN/m^2)	Snow load (kN/m^2)
Volta slab*	5.00	2.00	
Wall including covering*	2.50		
Roof slab	2.00		0.75 (1.00**)
Cover material of roof	1.00		

* Density: 20 kN/m^3.
** Snowdrift load.

Table 7
Analysis parameters [33].

Parameter	Definition	Value
A_o*	Effective ground acceleration coefficient	0.40 g
I	Building importance factor	1.0
$T_{x,y}$**	*x*'th and *y*'th natural vibration periods of building	T_x = 0.23 s, T_y = 0.25 s
T_A, T_B***	Characteristic ground periods	T_A = 0.15 s, T_B = 0.60 s
S (T_1)	Spectrum coefficient	2.5
R_a (T_1)	Seismic load reduction factor	2.0

* For seismic zone I.
** Analysis result.
*** For Z3 local site class.

The structure examined by using mode combination method was calculated under earthquake loads; all acceptances and obtained values were determined in accordance with the Turkish Earthquake Regulation (Table 7).

4. Numerical analysis

4.1. Structural system model

A computerized model of the examined building was prepared. In the mentioned model, walls and plates that are horizontal struc-

Fig. 2. Building performance levels: (a) Operational, (b) Immediate occupancy, (c) Life-safety, (d) Collapse prevention level [34].

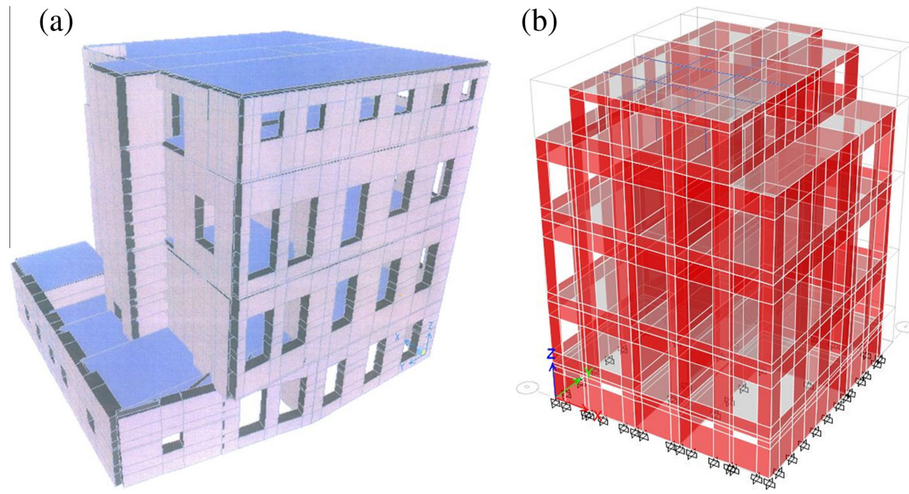

Fig. 3. Finite element model of the examined structure: (a) 3D view of the building in pre-restoration, (b) Finite element model of the building.

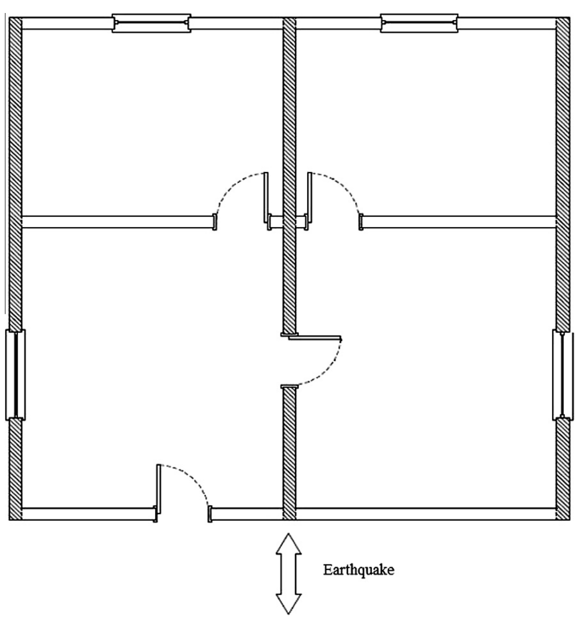

Fig. 4. Masonry walls in storey plan, l_d: total length of the shaded area, A: gross storey area [33].

tural members were modelled as shell element in the element library of the finite element analysis software. Mechanical characteristics of the structural members in the computerized model were transferred to the structural system by means of the data obtained from the experimental studies mentioned above. 3D view of the building and its finite element model of load-carrying system are given in Fig. 3.

4.2. Analysis of the existing load-carrying system using the linear elastic method

4.2.1. Compliance of the structure with the regulation

In determining the structure's performance, controls were made based on the Turkish Earthquake Regulation [33]. A three-dimensional linear finite element analysis of the four-storey masonry building under seismic loading is carried out.

Control of the total maximum length of the masonry walls is given in Eq. (1).

Table 8
Total maximum length control.

Storey	l_d (m)		Area, A (m^2)	l_d/A (m/m^2)		$l_d/A \geq 0.2\ I$	
	x-dir.	*y-dir.*		*x-dir.*	*y-dir.*	*x-dir.*	*y-dir.*
Ground	56.03	57.87	272.54	0.212	0.206	√	√
First	32.35	40.68	221.33	0.146	0.184	×	×
Second	23.77	47.96	221.33	0.107	0.217	×	√
Attic	40.62	47.54	221.33	0.184	0.215	×	√

$$l_d/A \geq 0.2 \times I\ \text{m/m}^2 \tag{1}$$

where l_d is the total supported wall length (m) regardless of window and door gaps in any storey; A is the gross storey area (m^2) and I is the building importance coefficient. Schematic representation of the stated parameters is given in Fig. 4.

Findings of the total maximum length control of the masonry wall in the examined building are presented in Table 8.

Non-supported length (l_i) between the vertically connected masonry wall axes to the plane of any masonry wall is limited to maximum 5.50 m in first-degree earthquake zone (Eq. (2)). Non-supported lengths in the normal storeys- and attic-plans are given in Fig. 5, respectively. Controls carried out according to the indicated limits are given in Table 9.

$$l_i \leq 5.5\ \text{m} \tag{2}$$

Limit conditions in door and window gaps in the walls are given in Fig. 6. Accordingly, distance from the building's corner must be at least 1.5 m and the distance between windows must be at least 1 m in the first-degree earthquake zone. Wall dimension controls for normal storeys are given in Table 10.

It was determined that the distance of door and window locations from the corner of the building did not always meet the control value, but the distance between the windows meet the condition values. It was pointed out that the window gaps were 4.16, equal to the non-supported wall length, and exceeded the limit condition value 3.69.

It is stated that each storey can be maximum 3 m in masonry buildings. Examining the storey heights of the building, 3.75 m, 4.84 m, 4.48 m and 3.10 m for the ground-, first-, second-floors and the attic respectively, exceeding the 3 m limit. According to the regulation, supported walls in masonry buildings should be as regular as possible and arranged to be symmetrical or close to symmetrical to the main excess, construction of partial basement must be avoided. When the structure is examined based on these

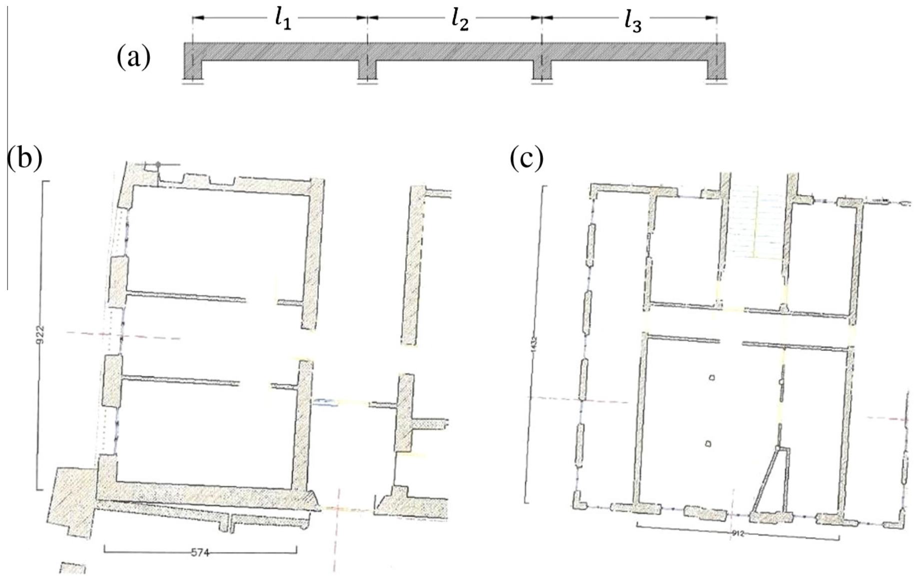

Fig. 5. (a) Non-supported wall length ($l_i \leqslant 5.5$ m), (b) View of non-supported wall lengths in normal storeys, (c) View of non-supported wall lengths in attic [33].

Table 9
The control of non-supported wall length.

Storey	Non-supported length (m)		Limit value (m)	Limit value control $l \leqslant 5.50$ m	
	x-dir.	*y-dir.*		*x-dir.*	*y-dir.*
Ground, first, second	5.74	9.22	5.50	5.74 > 5.50	9.22 > 5.50
Attic	9.12	14.32		9.12 > 5.50	14.32 > 5.50

criteria, it is determined that the structure is not regular on the plan and has diagonal masonry wall members.

4.2.2. Modal analysis

Twelve modes were calculated for the building in modal analysis and period values corresponding to the said modes are presented in Table 11. Participation mass ratio and period values are examined and it was found out that the vertical period values is 0.46 s and horizontal period values is 0.34 s. Data presented in Table 11 demonstrates that the horizontal direction is more rigid than the vertical direction.

4.2.3. Shear stress control of the wall

For each earthquake direction in the regulation, by proportioning the horizontal force coming to the horizontal load-carrying members to the wall's horizontal section area, shear stresses on the walls were obtained. Calculated values were compared to the experimental shear stress values. Shear stresses calculated individually for each storey and every wall across the building's height, were observed to be smaller than the shear stress. Total shear power calculated for finite elements analysis for each direction, by proportioning the total section area in that direction, principal stress comparisons were made on the shell elements together with shear stress for each storey.

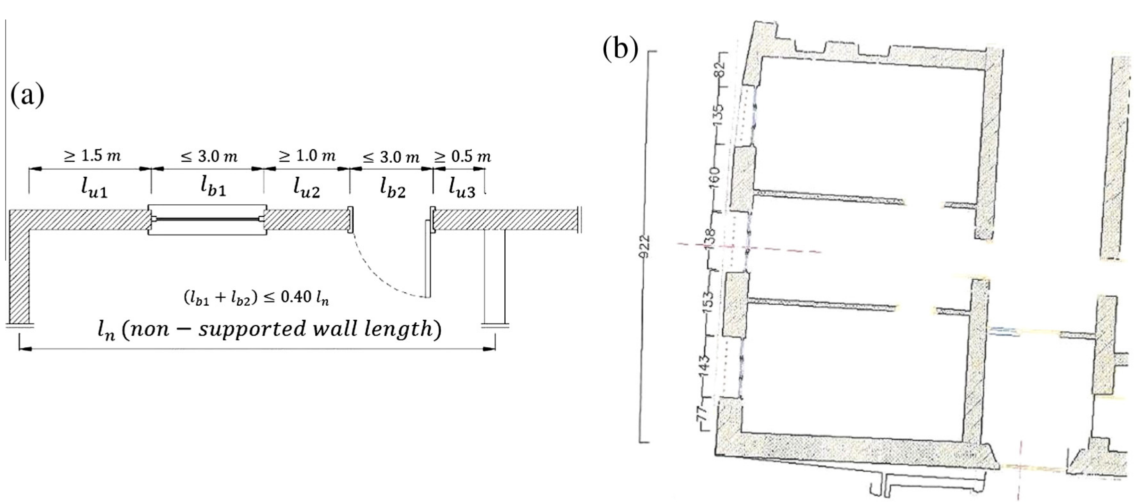

Fig. 6. (a) Limit conditions of windows gaps and locations for the first-degree earthquake zone, (b) Window dimensions in the storeys [33].

Table 10
Control of wall dimensions and window gaps.

Storey	l_{u1}	l_{u2}		l_{u3}		l_{b1}	l_{b2}	l_{b1} + l_{b2}
	⩾1.5 m	⩾1.0 m		⩾0.5 m	⩽3.0 m			⩽0.40 l_n = 3.69
Ground, first and second	0.77	1.53	1.60	0.82	1.43	1.38	1.35	4.16

Table 11
Period values corresponding to 12-modes.

Mode	Period (s)	UX*	UY*	UZ*	Total X**	Total Y**
1	0.46	0.59	0.00	0.00	0.59	0.00
2	0.34	0.01	0.61	0.00	0.60	0.61
3	0.27	0.01	0.04	0.00	0.61	0.65
4	0.25	0.00	0.00	0.03	0.61	0.65
5	0.22	0.00	0.00	0.03	0.61	0.65
6	0.21	0.00	0.00	0.02	0.61	0.65
7	0.20	0.00	0.00	0.03	0.61	0.65
8	0.20	0.00	0.00	0.00	0.61	0.65
9	0.19	0.00	0.00	0.08	0.61	0.65
10	0.18	0.00	0.00	0.08	0.61	0.65
11	0.17	0.02	0.00	0.01	0.63	0.66
12	0.17	0.01	0.00	0.00	0.64	0.66

* Modal participation mass ratios in x-, y-, z-directions.
** Total modal participation mass ratios in x-, y-directions.

Storey shear forces obtained from the structure analysis software were compared to the forces that were experimentally obtained and shear stress controls were carried out in the structure. Shear performance on x and y directions is presented in Table 12.

Shear strength of all the walls in both directions of the masonry building was determined to be adequate to cover the shear forces under earthquake effects. It was determined that the controls did not meet the anticipated rules for the building geometry and design.

In analysis carried out with general purpose finite element analysis software [32], it was determined that partial stress concentrations occurred in circumference of irregular door or window gaps on the south facade. The said stress concentrations occurring under horizontal effects was anticipated not to cause total migration across the building; meet partial crack and immediate use performance level criteria; and not to transform into the state of migration.

5. The retrofitting applications of the structure

As a cross-disciplinary study must be carried out in strengthening stage of the historical building, improved or changed details were manufactured with the approval of engineering and architecture disciplines. Strengthening stage included: i. Injection application to wall members, ii. Detailing including stainless tie members of the volta slabs (brick floor arch), iii. Detailing door and window gaps by using steel plate, iv. Adding sections by using carbon fiber wrap in exterior walls and anchorages in the interior walls.

Table 12
Wall shear stress control in each storey.

Storey	$\sigma_{wall,analysis}$ (MPa)		$\sigma_{wall,test}$ (MPa)	Limit value control ($\sigma_{wall,analysis} \leqslant \sigma_{wall,test}$)	
	x-direction	*y*-direction		*x*-direction	*y*-direction
Ground	0.244	0.377	0.40	√	√
First	0.208	0.387		√	√
Second	0.311	0.184		√	√

5.1. The strengthening of the walls using injection technique

After completing plaster removal on all masonry walls, structural cracks were determined. In order to repair them according to the building's structure, mixture ratio determined in the laboratory for the samples taken was used. The said mixture was applied to the cracks by means of manual injection. Integration was ensured by using steel clamp members in larger cracks and anchorage elements on the walls were anchored by means of epoxy (Fig. 7).

Since it takes time to settle the injection compound into the wall, application was repeated on different times. The compound inside the pipe was penetrated into the interior walls in time, the pipes were continuously injected and this process continued until the gaps were filled.

5.2. The strengthening of the brick floor arches

One of the primary interventions in the structure is strengthening of the brick floor arches. This application consists of six steps:

- Covering on the brick floor arches were removed and the existing I profiles were revealed. Parts exposed to minor corrosion on the original volta slab steel profiles were cleaned, painted with double layer antirust paint and 10/40 mm lama profile was fixed by means of welding I profile to the top cover (Fig. 8a). The purpose of the lama profile fabrication is that the shear steel bar's welding thickness and surface are inadequate on the existing profiles.
- Ribbed shear steel rebar of ϕ10 was attached on the volta slab beams by welding (Fig. 8b).
- Using ϕ20 stainless steel tie members, masonry walls on the horizontal plane were integrated (Fig. 8c–e and g). The mentioned system passed through the masonry walls and fixed with 100/200/10 stainless plate and M20 bolt (Fig. 8f). Tensioning the stainless steel system was achieved by means of a geared system detail (Fig. 8g).
- Holes were drilled with a drilling core of ϕ14 and filled with epoxy in masonry walls. Steel rebar of ϕ8 was inserted in the drilled hole to support before the epoxy was set (Fig. 8d).
- Masonry walls with ϕ8 steel rebar anchorage and ϕ10 shear steel rebars fixed on the brick floor arches, wire mesh with ϕ 8/20 were assembled. Thus, integration was ensured between the steel profiles in the brick floor arches, masonry walls and ribbed welded wire mesh system (Fig. 8e).
- After having completed the mentioned reproductions, topping concrete was applied on the volta slab (Fig. 8h).

5.3. The strengthening of the gaps of doors and windows using steel plates

Original window and door dimensions were based on the restoration application, all the windows that has been cancelled out on the facades were opened and as a result of 3D analysis results, it was decided to strengthen the window and door gaps with steel plate to increase the strength in order to resist static and dynamic effects. In this regard,

Fig. 7. Application of wall injection: (a) Examination of all the walls after the plaster removal, (b) Determination of structural cracks, (c) Preparation of the surface for injection, (d) Completing injection application with manual technique.

Fig. 8. Strengthening of the brick floor arches: (a) Removing top layers of the slab, adding ϕ10 shear rebar on the slab steel profile beam and painting with antirust paint, (b) Assembling shear rebars on the steel beams in the volta slabs, (c) Steel tie fabrication, (d) Planting the ores on the masonry walls, (e) Completing the welded wire fabrication, (f) Completion of the connections of stainless steel ties between the masonry walls, (g) Stretching the stainless steel profiles, (h) Topping concrete application on the slab.

Fig. 9. Application of steel plate profiles on door and window gaps: (a and b) Composing a surface with polymer modified mortar with 3 cm thickness grout additive, (c–f) Steel plate assemble on door and window edge surfaces, (g) Preparing the surface for plaster application, (h) Plaster reproduction to conceal the steel plates.

- Copolymer primer was applied on masonry brick surfaces in window and door edges in order to create suitable surface for steel plate, afterwards, polymer modified mortar with 3 cm thickness grout-additive was applied (Fig. 9a and b).
- Steel plates of 8 mm thickness were assembled on the interior edges of door and window gaps (Fig. 9c and d).
- Anchorage holes were drilled with a drilling core of $\phi 12$ and filled with epoxy. Stainless steel rebar of $\phi 10$ was inserted in the drilled hole to support. Steel plate monolithic connections was made with stainless bolts (Fig. 9e and f).
- Steel plate surfaces were covered with epoxy and quartz sand was sprayed to make them ready for plaster application (Fig. 9g).
- Finally, plaster was applied to the steel surfaces and steel plates were hidden underneath the plaster (Fig. 9h).

5.4. The strengthening of the facade walls using carbon fiber wrap

Following exterior facade plaster removal, injection was applied to improve the cracks. Carbon fiber wrap with 0.3 m width was used in tensile stress areas on the wall elements in order to resist the horizontal effects such as earthquake. Fig. 10 presents the applied carbon fiber locations on the facades. Carbon fiber wrap was positioned so that it would not affect window gaps and storey levels and concealed under lime-based plaster.

Carbon fiber application phases are given below:

- Application of copolymer primer on the existing bricks.
- Following the primer application, creating a surface suitable for carbon fiber wrap with polymer modified mortar with 3 cm thickness grout additive.
- In preparing the surface, epoxy-based saturating resin of 1.2 kg/m^2 was applied, following, strengthening layer is applied using carbon fiber wrap.
- Based saturating resin of 0.7 kg/m^2 was reapplied on carbon fiber wraps and preparing the surface for exterior plaster by applying quartz sand.

The views of aforementioned all the application stages is given in Fig. 11.

5.5. The strengthening of the internal walls using ore plantation technique

After having completed the existing plaster removal from the vertical load-carrying members, internal wall strengthening was applied. In this respect;

- Four anchorage holes per square meter were drilled on masonry walls and filled with epoxy (Fig. 12c–e).
- 4ϕ14 steel rebar ore per square meter were planted on the masonry walls related to exterior facade and 4ϕ12 steel bar ore per square meter were planted on the walls between rooms and corridors (Fig. 12a).
- Following the steel rebar anchorage, mechanical integration was ensured with existing steel rebar anchorages of welded wire meshes of ϕ131/131 (Fig. 12c–e).

Following masonry wall steel rebar assemblies, plaster reproduction was carried out and mixtures prepared as a result of plaster analyses were used.

In strengthening the internal walls, total plaster thickness was 6 cm, consistent with the original plaster thickness.

6. Conclusions

In restoration applications carried out to pass the historical masonry buildings on to new generations, it is decided to optimally

Fig. 10. Location of carbon fiber wrap application on four facades: (a) Southern facade, (b) Western facade, (c) Northern facade, (d) Eastern facade.

Fig. 11. Exterior facade application of carbon fiber wrap: (a and b) Detail of polymer modified mortar with 3 cm thickness, (c–f) Preparation by spraying quartz sand to make it ready for plaster.

preserve such buildings' original forms and to make interventions to increase the building's service life; in this regard, it is important to preserve the structures' historical identity and constructional value. In this study, strengthening applications carried out in line with laboratory and numerical analyses to increase the building's strength in order to resist vertical and horizontal load effects by

Fig. 12. Strengthening the internal walls with ore plantation method: (a) Strengthening locations of the first- and second-floor walls (interior), (b) View of the wall after plaster removal, (c–e) Strengthening the wall by using wire mesh, (f) Using polypropylene fibers in plaster content, (g) Application of thick plaster, (h) Application of thin plaster.

means of various engineering and architecture disciplines during restoration of a historical building located within Istanbul University were presented. The mentioned study was aimed to become an exemplary study in strengthening applications to increase historical buildings' seismic resistant.

Results obtained within the scope of strengthening masonry structures are summarized as follow:

- Wall plasters were removed to determine the existing strength of all internal and external facade walls; in case of structural cracks, injection method was used to fill these cracks. Also clamp was applied based on the crack's geometry.
- Steel profiles remained inside the slab generally in brick floor arch-type masonry structures may partially lose their characteristics or rigidity problems may occur in a composite slab member. On the other hand, non-rigid brick floor arches (volta slab) cannot be used to prevent various displacement of masonry walls under horizontal effects.
- Although it increases the building's weight, cement addition instead of materials included in the brick floor arches, except the steel profiles, has an impact on the exposure of the walls to compression effect, provided that horizontal rigidity is ensured.
- Lama profile was added on I profiles with welding to increase the profile strength in volta slabs. In order to ensure plane rigidity of storey slab, anchorage holes were drilled on masonry walls and integration was achieved with welded wire mesh addition.
- Within the scope of 3D numerical analyses, non-continuous stress concentration from top floors to the foundation was determined in partial walls and such concentration was removed with local precautions. In this scope, x-form carbon fiber wrap was applied in window gaps to increase the strength of external walls in order to resist the earthquake effect, in order to prevent the stress concentration on the south facade and the negative impact on the building's usage functions. Also, surfaces of window edges were strengthened with steel plate and structural cracks were repaired with injection method.
- As the walls serve as load-carrying members in masonry buildings, anchorage holes were drilled on the internal walls and steel bar addition was made on vertical and horizontal directions.

As a result of the analyses carried out under the identified design earthquake, it was determined that the existing system meets the conditions set forth for life safety performance level. For an earthquake with 10% recovery possibility in 50 years, it was determined that the shear strength of all the walls in both directions of the masonry building was generally adequate to meet the shear forces occurring under the applied earthquake effects. In this case, it can be said that the building meets that "immediate usage performance level", however, due to insufficiencies not meeting the masonry building definitions of the structure and partially determined irregularities, "life safety performance level" should be used to evaluate, rather than "immediate usage performance level".

It was suggested that the intervention techniques including the repair and strengthening approaches presented within the study's scope could apply to historical masonry structures, as a result of backing up lab tests and numerical analyses. Also, it was anticipated that the repair and strengthening application on the structure, especially in case dynamic effects, such as earthquakes, are concerned, would be improved to increase structural performance and strength. Architectural interventions expected to be carried out in scope of the restoration applications, are outside the scope of this paper but included in the restoration process.

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