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RESEARCH ARTICLE

Finite Element Analysis of a Historic Masonry Building with Unique Architectural Features: A Case of KLP Building at İÜC Cerrahpaşa Campus

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ABSTRACT
The walls and slabs of historic masonry buildings, as load-bearing members, tend to lose their strength and durability over time, which can lead to poor seismic performance. Rehabilitation and strengthening practices are performed to protect historic masonry structures and increase their lifespan. On the other hand, before a faithful restoration practice, assessing the seismic performance is important to perform strengthening work during restoration. With this purpose, this study presents a computational approach for seismic performance examination of a historic masonry building with unique architectural features constructed in the early 1930s. The study was composed of two stages; a historical background examination and a computational examination based on numerical analyses. The historical examination includes addressing all functional changes in the building from the construction date to the present. The numerical analyses involved preparing a three-dimensional finite element (FE) model based on the current state of the building and subjecting it to seismic loading. The analysis results revealed that the building is not seismically safe. The study is believed to provide a valuable contribution to the literature by presenting a numerical seismic assessment of the masonry building for consideration in the restoration process.

Keywords: Masonry buildings, Seismic performance analysis, Structural safety

1. Introduction

The primary goal of a repair or retrofitting intervention is to decrease potential sources of damage, reintegrate the building into society, or prevent further damage to the building. Since earthquake loads affect a building according to its weight and rigidity, increasing the rigidity and weight of a masonry building should be avoided during a repair or retrofitting practice (Çöğürcü, 2007). Currently, there is no distinct seismic code in Turkey that specifically addresses the structural assessment of historic buildings. Moreover, the evaluation of registered historic and cultural heritage structures and monuments is not encompassed in the Turkey Building Earthquake Code (TBEC, 2018). Therefore, historic buildings are subject to the same seismic code as other regular buildings due to the lack of a specific code for evaluating the seismic safety of historic structures. However, evaluating historic buildings requires a more complex approach than that for normal buildings, as various additional factors need to be considered. Most historic buildings were constructed using construction techniques and methods that were available during their time period. However, the current seismic code assesses the seismic performance of masonry buildings based on shear forces. On the other hand, performance assessment by considering only shear forces is an inadequate method for historic masonry buildings. To accurately evaluate the seismic performance of historic masonry buildings and propose effective retrofitting measures, it is necessary to consider building displacements under various loads, including dead loads, live loads, and earthquake loads, as well as the stresses on structural elements. New buildings constructed according to current seismic codes and standards are generally assumed to have a certain level of performance. However, the situation is different for historic buildings. Therefore, each historic structure should be examined for seismic performance using a proper and efficient method considering its architectural and historical significance.

Many studies on seismic performance assessment of masonry buildings were reported in the literature. Koçak (1999) presented some practices carried out for the protection and retrofitting of historic masonry buildings. A finite element model of the Little Hagia Sophia Mosque was developed and analyzed to determine dynamic features. The analytical and experimental results were

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compared. In another study, Sallio (2005) examined a masonry hospital building constructed in 1950 and found that the current state of the building has poor shear strength. A retrofitting proposal was presented, and the current and retrofitted states of the building were analyzed and compared. Taliercio and Binda (2006) presented a structural analysis of the Byzantine Basilica of San Vitale using a finite element model. Due to the complex geometry of the structure, linear elastic analysis was conducted based on the simplified rules. The results showed that the building could not carry its own weight. The findings obtained by the displacement measurements conducted with 6-month intervals matched with the performance analysis results. Özen (2006) modeled a masonry building in Hasankeyf, Turkey using the finite element method. The author performed linear and nonlinear analyses and compared the results. Both analyses showed that the damages were on the same section of the building. Therefore, they consider linear analysis is sufficient since nonlinear analysis takes more time. Betti and Vignoli (2008) examined static and dynamic -linear and nonlinear- analysis of historic masonry buildings and presented a seismic performance assessment of a basilica under earthquake loads. Strengthening suggestions were presented and compared based on the potential damage scenarios obtained by the analyses. Roca et al. (2010) discussed the practicality of limit analysis, simplified methods, finite element method, macro or micro modeling, and discrete element method for structural examinations. Ercan (2010) performed an operational modal analysis on two masonry buildings. The dynamic data obtained by experimental and numerical analyses were compared. The safety level of the buildings was assessed by static and dynamic analysis. Branco and Guerreiro (2011) compared the performances of various seismic strengthening techniques on an exemplary masonry building model. Some methods including reinforced concrete (RC) walls and viscous dampers were discussed. These methods were compared considering maximum displacements and stresses that occurred on the masonry walls under earthquake effects. Temur et al. (2013) examined earthquake risk analysis results of 57 different buildings. Due to some reasons including the number of examined buildings, limited time, as well as the complex geometry and irregular layout of the buildings, they employed the Rapid Assessment Method. Using this method, they determined the risk level of the buildings rapidly. Soveja et al. (2013) conducted a study addressing the complexity of the building and choosing the right modeling method for the analysis and strengthening of historic masonry buildings. They discussed the pros and cons of different modeling methods and evaluated their practicality. Yıldızlar and Akçay (2018) examined the current status of an educational building and assessed the safety level of the building using the Rapid Assessment Method before a strengthening practice. They also examined the effect of wrong interventions on the structural elements on the safety of the building.

Some modeling techniques are available for seismic analysis of historic masonry buildings. These are the simple wall model, crossbar model, equivalent frame model, and finite element modeling methods. Among these methods, finite element modeling is the most common method. The finite element modeling method involves the use of equations that describe the stress-displacement relationships of structural mechanisms. The program allows rapid assessment of complex analyses. There are various analysis methods available for evaluating the seismic performance of historic masonry buildings, including linear, non-linear, and time history analyses. Among these methods, linear analysis is the most common method. Linear analysis can employ the equivalent earthquake loading method. The design spectrum is determined according to the location of the examined building. The spectrum is then defined in the analysis program as the input for the earthquake load. The current seismic code stipulates that for each mode, the minimum ratio of total mass that effectively meets the base shear force to the total building mass should be 95% (TBEC 2018).

An extensive literature review indicated that numerous studies have been conducted on the seismic behavior assessment of historic masonry buildings. The reviewed studies addressed various aspects of assessing the seismic behavior of historic masonry buildings, including the determination of material properties of load-bearing members, comparison of linear and nonlinear analysis methods, kinematic analysis, time history analysis based on real earthquake records, and evaluations according to seismic code. The current study aims at assessing the seismic performance of a historic masonry building and evaluating the results according to the current seismic code. For this purpose, a finite element model of a building was prepared, and the seismic performance of the building was determined to inform restoration practices.

2. The examined building

The current building is a masonry style building with brick walls, consisting of two floors, has a closed area of 447 sqm (Figure 1). It is located within the campus of Istanbul University-Cerrahpaşa Faculty of Medicine in Fatih district, Cerrahpaşa neighborhood, Block 1138, and Plot 1. While still standing, the building has suffered the loss of many of its original and distinctive architectural element. The building has been utilized as the faculty of medicine for a considerable period. While the building was initially constructed to meet the faculty's classroom needs, it was used for various other purposes over time. The building was used as an amphitheater, pathological anatomy institute, x-ray unit building, and finally Consultation Liaison Psychiatry Department in the past. Recently, the building was closed down and abandoned.

After the *Istanbul Darülfünun* was closed and Istanbul University was established, the Faculty of Medicine in *Haydarpaşa* was moved to *Beyazıt* in 1933. Then, the central building of the Faculty of Medicine, administrative departments, institutes, and laboratories were moved to the War Office (*Harbiye nezareti*) building. The clinics of the Faculty of Medicine were distributed to

Figure 1. The current condition of the examined building

five different hospitals in Istanbul. On the other hand, Surgery, First Internal Medicine, and Eye clinics were moved to *Cerrahpaşa* Hospital (Figure 2).

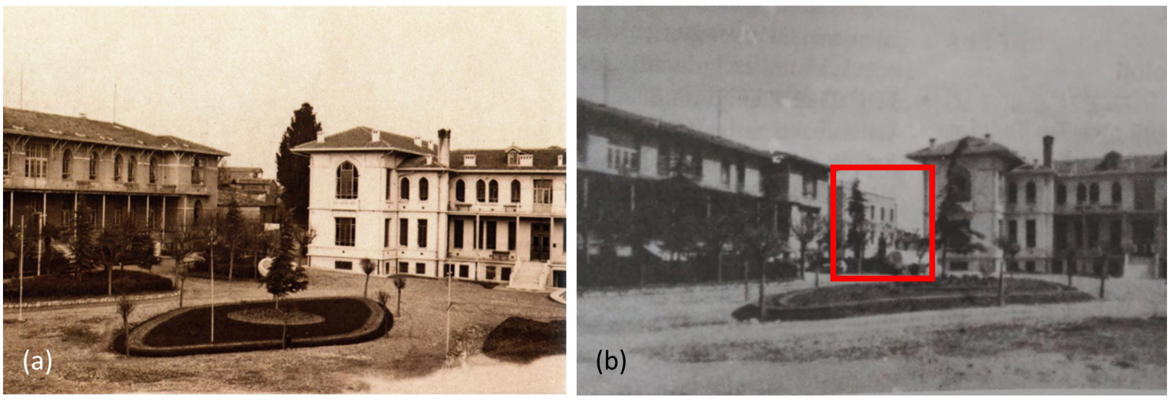

Figure 2. The examined building (a) Before construction, Internal Medicine and Surgery Clinic (Altıntaş, 2011), (b) After construction (Sarı et al., 2009)

Cerrahpaşa Hospital, which operated under the municipality in the past, has started to operate with the Istanbul University Faculty of Medicine since 1933. During the academic year of 1933-1934, the Faculty of Medicine had 1360 students, and around 180 students took courses specifically at *Cerrahpaşa* Hospital. In order to meet the increasing demand for educational activities and the insufficient capacity of the hospital, a classroom building was constructed between the Internal Medicine Clinic (Currently the Dean Building) and the Surgery Clinic (Currently the Psychiatry Clinic) as shown in Figure 3.

The construction of this building was completed in 1933 and was the first classroom building in *Cerrahpaşa*. Plus, it was the first building constructed by İstanbul University in *Cerrahpaşa*. After its construction, the building was referred to as the *Cerrahpaşa* Classroom building, but it was later renamed the *Neşet Ömer* Amphitheater to commemorate Rector Neşet Ömer İrdelp's death in 1948 (Altıntaş, 2011). The current status of the amphitheater is shown in Figure 4.

The building was constructed in masonry construction style using bricks. The load-bearing walls were built using clay bricks and mortar composed of a hydraulic lime-aggregate mixture (Figure 5). Slabs - which is an important component of the load-bearing system- were built as RC slabs. The utilization of RC slabs instead of the more common jack arch slabs, frequently employed in historic masonry buildings, suggests that the construction of this building differs from other similar structures of the same period.

2.1. Original plan features

A restitution report was prepared to identify the original architectural features of the building (Proger S.p.A & ZH Co. 2013). Based on the findings of the restitution report, the building consists of a ground floor, a standard first floor, and a standard second floor, all constructed using masonry brick walls in a masonry style (Figure 6). Also, the building has a hip roof. In later times,

Figure 3. The original building in 1930-1933 (view from the southwest)

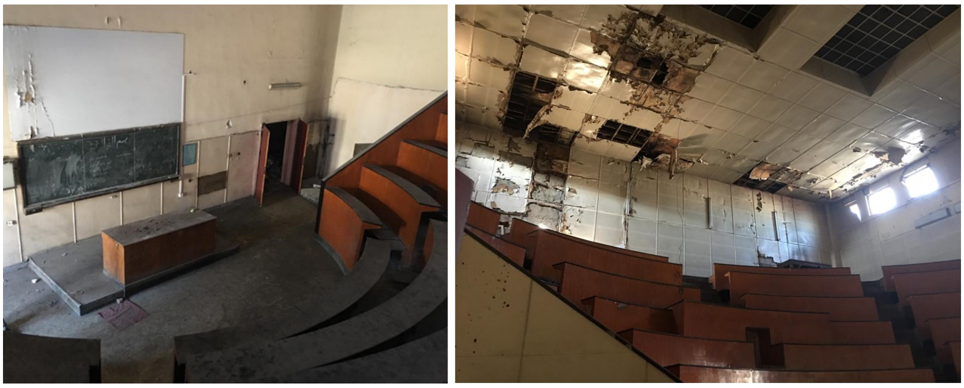

Figure 4. The current status of the amphitheater

unqualified interventions were made to the roof. Marseille tiles were used in the roof covering. Gable walls were used around the roof, next to the slope of the roof. The top of the amphitheater classroom was illuminated by sunlight.

The main entrance of the building is located on the south façade. "The masonry brick facades of the building were covered with a layer of cement plaster, followed by an RC-based material and paint.

Another RC building was constructed in later times next to the south façade of the examined building.Shared spaces were created between the examined building and the consultation-liaison psychiatry building. The main entrance is accessed through an iron double-door that appears to have been improperly installed during a later period of the building's history. Some improper connections were made to link the building with surrounding buildings. An RC tunnel (Figure 7) was added to link the building to an adjacent structure, with a door opening to the neighboring building (the Dean building). While the ground floor of the building remained within its original borders, expansions were made to some areas on the first and second floors. On one of the façades has two separate entrances, with the first providing direct access to the largest section of the building, the amphitheater, through a double-winged iron door added at a later time. The second entrance, which is located on the ground floor, was created by converting an original window opening into a doorway.

During the restoration process, the wall plasters were removed to partially uncover the connections between the structural elements, revealing the presence of horizontal and vertical RC peripheral ties in some of the walls. It should be noted that not all walls had horizontal and vertical RC peripheral ties. As shown in Figure 8, in some sections wall-slab joints were through horizontal RC peripheral ties, while in other sections, RC slabs were directly connected to the masonry walls.

Figure 5.(a, b) Masonry walls, (c) RC Slab

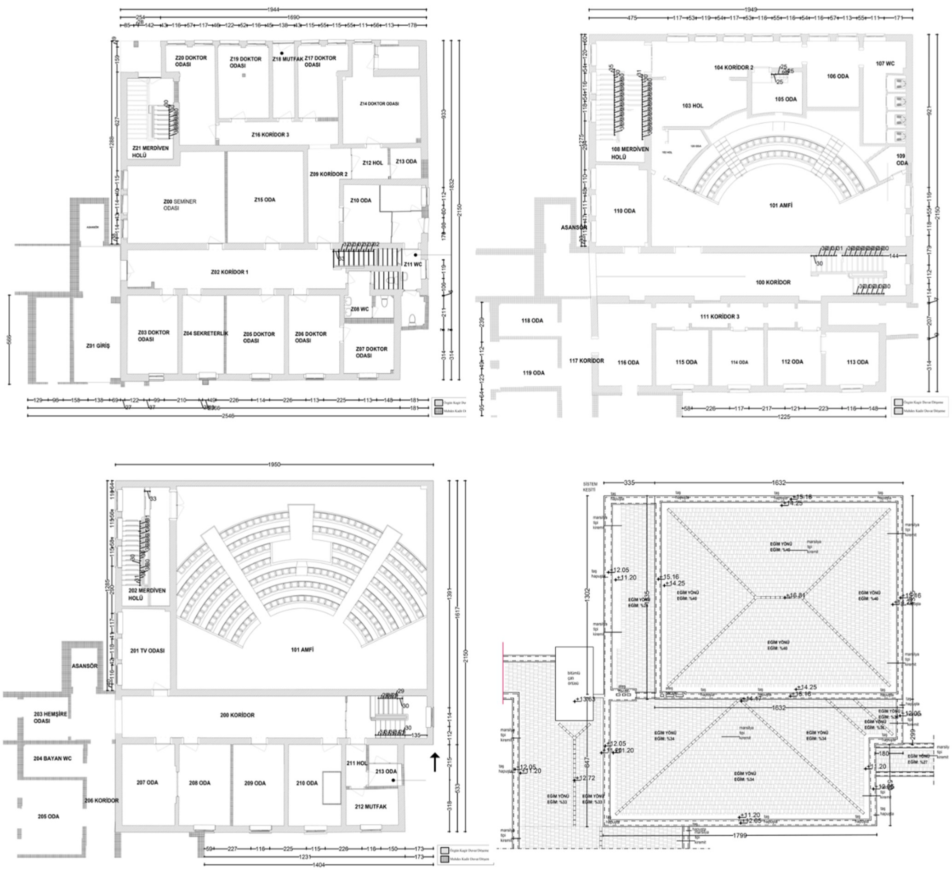

Figure 6. Plan views of the ground, 1st, and 2nd floors

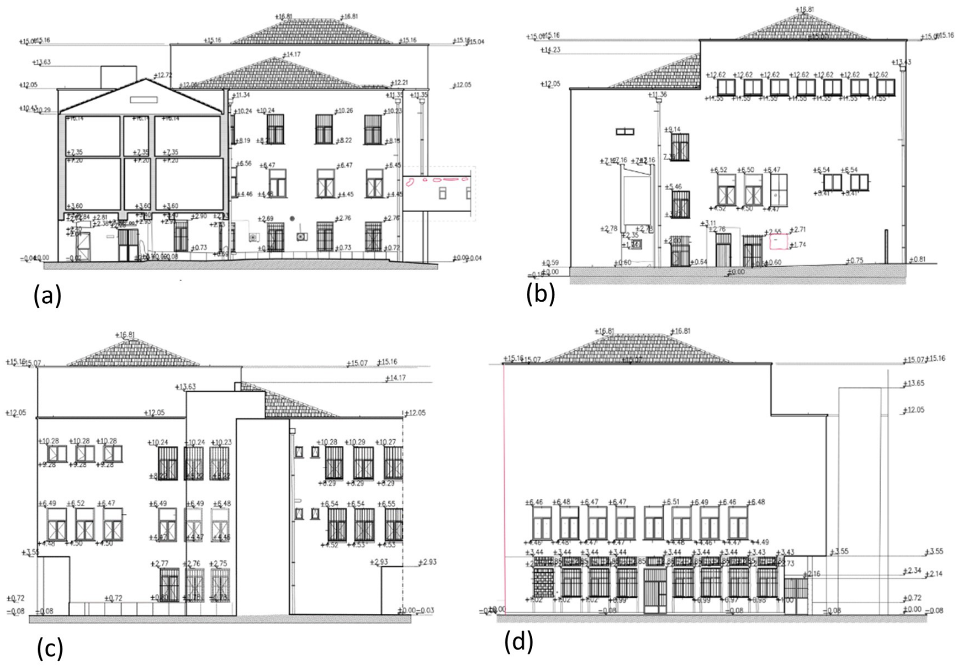

Figure 7. Façade views of the examined building. (a) South, (b) East, (c) West, (d) North

Figure 8. Connection details between the structural elements

2.2. Interventions made to the Load Bearing System

The examined building has undergone numerous interventions throughout its history, resulting in the loss of several unique components.

The interventions carried out on the building aimed to adapt it for a change of use or to repair and strengthen it following damages caused by seismic events. Examples of revisions to the building's architectural plan due to changes in use include the cutting of window and door openings in the load-bearing walls (Figure 9). After scraping the wall plasters, some openings were discovered which had negative effect on the load-bearing capacity of the walls. To fill these openings, new perforated bricks were used. On the other hand, steel columns and beams shown in Figure 10 are examples of the strengthening interventions performed in the past.

Figure 9. An opening cut in the masonry wall (later closed)

Figure 10. Steel columns and beams added to the building

These steel columns and beams were discovered after removing suspended ceilings and demolishing later-added walls.The most significant interventions made to the building involved the addition of connections on both side facades, which were designed to provide access to adjacent buildings. The original plan of the building includes only one access to the Dean Building from the 1st floor. However, as seen in Figure 11, additional connections were later made to the ground and 2nd floors to connect with the Dean Building.

The original plan did not include a connection to the Psychiatry Clinic from the south façade. However, an RC structure was constructed in 1951 to connection the buildings as seen in Figure 12. This building was later used as the Department of Pediatric Mental Health and Diseases.

Figure 11. Current connection of the building to the Dean Building

Figure 12. Connection of the building to the Psychiatry Clinic

3. Numerical Analysis

A numerical model was developed to simulate the structural behavior of the examined building, considering its plan features. The model included one ground and two normal stories and was subjected to dead and live loads representative of the building's intended function. The building model underwent a seismic analysis, and its performance was evaluated based on the criteria outlined in TBEC 2018.

3.1. Modeling

The 3D finite element model of the examined building was created on the Midas Gen 2019 program. Masonry walls were modeled using three or four-node shell elements. To accurately represent the actual condition of the building, structural illegalities and geometric defects obtained by on-site observations were incorporated into the model. The floor plans were idealized on CAD software to create a 3D-rod model, which was then defined in the Midas Gen software. For seismic evaluation, the areas with maximum stress were chosen as critical sections, as seen in Figure 13. These sections were located below the window elevations on all floors. A 3D view of the model is shown in Figure 14.

In the numerical model, critical sections were represented as wall members with varying thickness, while all other members were modeled as plate elements. These sections were identified and coded based on their corresponding wall numbers, and the analyses were evaluated considering these sections. Slabs were modeled as plate elements, while wall-slab connections were modeled as fixed supports.

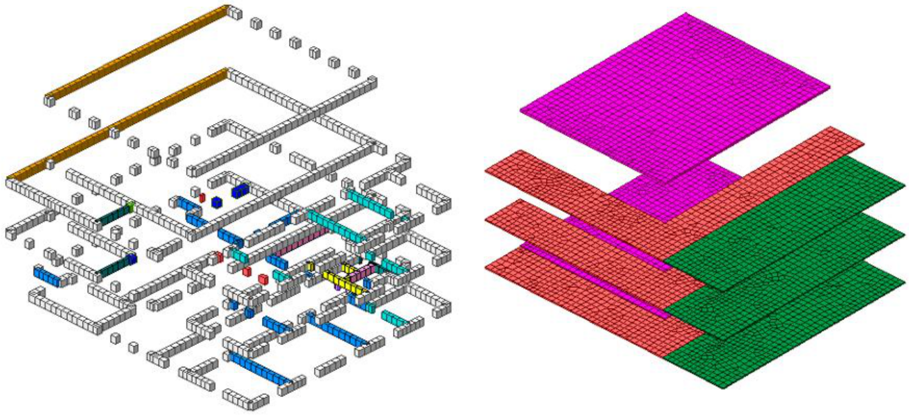

Figure 13. Critical wall sections and story slabs

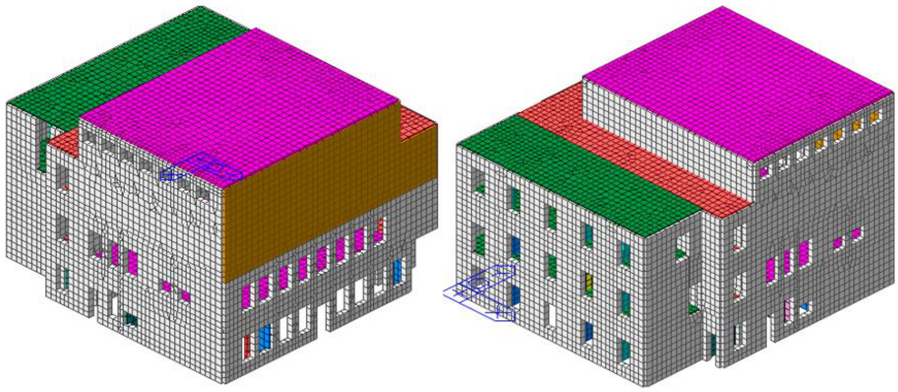

Figure 14. The finite element model

Dead and live loads were incorporated into the finite element model based on the specifications provided in the TS498 standards. After defining the structural behavior according to TBEC2018 in the program, earthquake loads were calculated using the earthquake load coefficient. The coefficient was calculated based on the mass participation ratio obtained from the modal analysis. These loads were then applied to the building in the numerical model. The parameters used in the static analyses are given in Table 1.

Table 1. Analysis parameters

Parameter	Value
Local soil class	ZD
Ground Motion Level	DD2
Earthquake Map Spectral Acceleration (g)	$S_s = 1.005$, $S_1 = 0.277$
Earthquake load reduction factor (Ra)	1
Building Importance factor (I)	1
Live Load Participation factor (n)	0.6
Seismic Design Class (DTS)	1a
Building Height Class (BYS)	6

3.2. Material Parameters

Material tests were carried out on the walls and reinforced concrete floors for the strengthening and repair works of the examined building. The report of the experiments was obtained from the Construction and Technical Department of Istanbul University.

However, these test reports do not include shear and compression tests for the walls. Only the physical characteristics of the bricks and binder materials were examined. For the reinforced concrete slab, only two cores were taken from the second floor, and a compressive strength test was carried out on them. The test results are shown in Tables 2 and 3.

Table 2. Test results for wall components (Özgünler, 2019)

Sample	Specific weight (kg/lt)	Water absorption % (by weight)	Binding/Aggregate ratio
Sample 1: Brick	2.53	26.8	"-"
Sample 2: Joint mortar	2.55	17.9	1 volume binder to 7.4 volumes aggregate
Sample 3. External plaster	2.57	11.1	1 volume binder to 5.2 volumes aggregate
Sample 4-1. Internal plaster	2.57	17.3	1 volume binder to 6.58 volumes aggregate
Sample 4-2. İnternal plaster (Dark colored)	2.48	48.2	1 volume binder to 5.28 volumes aggregate

Table 3. Cylindrical compressive strength test results for reinforced concrete slabs (Özgünler, 2019)

Core	Dimensions (diameter and length in mm)	Compressive strength (N/mm^2)
Core 1 (No 5)	93.2x94.0	22.1
Core 2 (No 6)	93.2x90.5	19.8
	Average	20.95

The modulus of elasticity of concrete used in the seismic analysis was calculated according to TS500 in line with the cylindrical compressive strength test. In seismic analysis, 85% of the average characteristic compressive strength of cubic samples was used. The modulus of elasticity was then calculated using Equation 1. Accordingly, the characteristic compressive strength of the concrete was calculated as 17.81 MPa and the modulus of elasticity was calculated as 27715 MPa. These values were considered in the seismic analysis.

$$E_{cj} = 3250\sqrt{f_{ckj}} + 1400 \ (MPa) \tag{1}$$

The specific weight used in the analysis was determined by calculating the average of the specific weights of the brick, mortar, and plaster material as seen in Table 2. Due to the unavailability of compression and shear test results, these parameters were obtained from the test report of a similar historical building with similar characteristics to the examined building. This reference building, the Istanbul University Faculty of Political Sciences Building, is used since it contains structural components representing different construction techniques. The test results of this building are summarized in Table 4. The average unit and characteristic compressive strengths were obtained from the brick and wall compressive strengths given in this table.

Table 4. Test results of the Istanbul University Faculty of Political Sciences (Akro Co, 2012)

Story	Test No	Compressive Strength of the Brick (Mpa)	Compressive Strength of the Wall (Mpa)
1	T1	4.803	2.402
1	T2	6.015	3.008
2	T3	5.579	2.789
2	T4	6.817	3.409
3	T5	5.455	2.727
	Average	5.7338	2.867

3.3. Soil and Earthquake Parameters

According to the V_{30} average shear-wave velocity mapping for the location of the examined building, the local soil class was determined as "ZD". TBEC 2018 envisages four different ground motion levels. In the seismic analyses, ground motion level-2 (DD-2) with a 10% probability of exceedance in 50 years (a return time of 475 years) was considered. Elastic acceleration spectrum characteristics were selected based on the soil class. The spectrum curve was drawn according to the local soil class given in TBEC

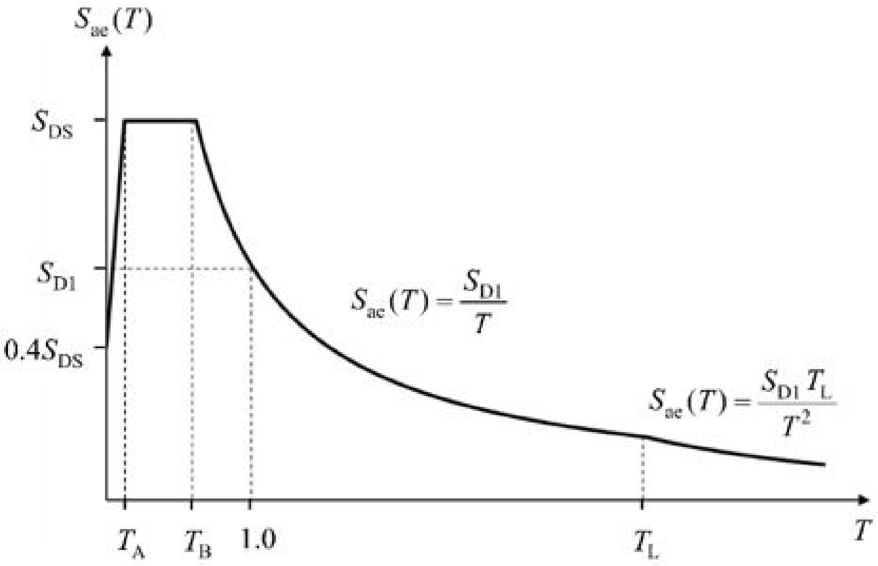

Figure 15. Spectral accelerations and lateral elastic acceleration spectrum

2018. Earthquake map spectral accelerations (S_s and S_1) were determined using the Turkey Earthquake Hazard Maps Interactive Web Application (AFAD, 2019).

Based on the location of the building on this map, spectral accelerations were determined as S_s = 1.005g and SS_1 = 0.277g for an earthquake with a return time of 475 years. Load soil factors were calculated as (F_s and F_1) F_s = 1.098 and F_1 = 2.046 according to the spectral accelerations and local soil class of "ZD".

Spectral accelerations were calculated by multiplying Earthquake map spectral accelerations with local soil class factor using Eq. 2.

$$\begin{aligned} S_{DS} &= S_S x F_S = 1.0051.098 = 1.103 \\ S_{D1} &= S_1 x F_1 = 0.2772.046 = 0.567 \end{aligned} \tag{2}$$

Accordingly, lateral elastic spectral accelerations [S_{ae}(T)] and lateral acceleration corner periods (T$_A$and T$_B$), were calculated using Eq. 2, the criteria given in TBEC 2018, and the elastic acceleration spectrum was created (Figure 15).

3.4. Structural Parameters

According to TBEC 2018, Ra was taken as 1 for the Mod Combination Method, which is the linear seismic analysis method used for the current building. The building importance factor (I) was considered in the seismic analysis of the examined building, and it was assigned a value of 1 according to TBEC 2018. The Live Load Participation Factor (n) was determined as 0.60 since the building was used for educational purposes.

Seismic Design Class (SDC) was determined as 1 based on the criteria outlined in TBEC 2018. Short period design spectral acceleration (S$_{DS}$) was calculated according to the Building Usage Class. Furthermore, according to the criteria given in TBEC 2018, Building Height Class (BHC) was taken as 6.

3.5. Performance Analysis Assumptions

The building information factor was chosen as limited information and accordingly, the building information factor was taken as 0.75. To assess the seismic performance, Mode Combination Method, a linear static calculation method was selected. The building should meet "Controlled Damage" performance criteria for a DD-2 ground motion with a 10% probability of exceedance in 50 years (return time of 475 years). For masonry walls; i) a building is classified as having "Limited Damage Performance" if the shear capacity of all walls meet shear demand caused by earthquake; ii) a building is classified as having "Controlled Damage Performance" if the ratio of walls in a given story that fail in shear is below 40%; iii) a building is classified as being in a "Collapse" state if the ratio of walls in a given story that fail in shear is above 40%.

4. Results and Discussion

For evaluation of the analysis results, SINAN software (Okumus, 2019) was used. The modes, natural vibration frequency, and mass participation ratios of the building were determined and are shown in Figure 16.

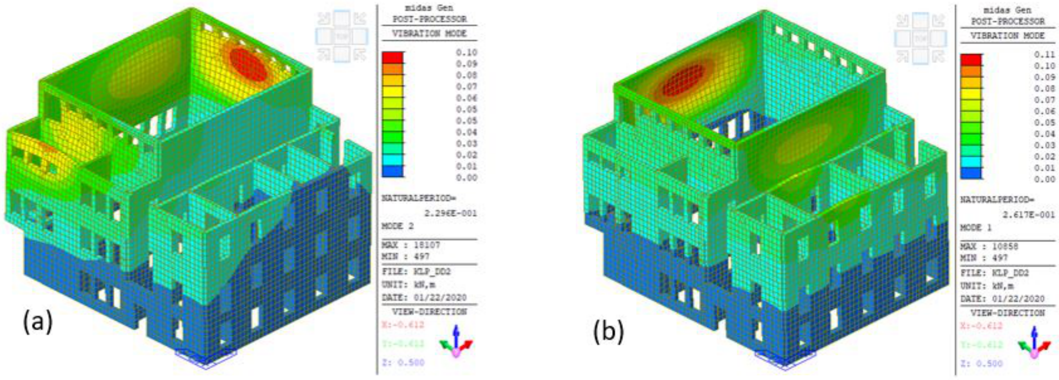

Figure 16.(a) Mode 1, T = 0.26 s, movement in the y direction, (b) Mode 2, T = 0.23 s, movement in the x direction

Modal analysis of the building under earthquake effects was performed. The number of vibrational modes required for the modal analysis was determined based on the criterion that the sum of total effective masses for each mode in the x and y directions should account for at least 95% of the building's total mass. Accordingly, the criterion was met for the first 35 modes in both x and y directions. The first dominant mode of the building was the 1st mode and the mass participation ratio and the period for lateral movement in the y direction were found to be 50.52% and 0.26 s, respectively. The second dominant mode of the building was the 2nd mode and the mass participation ratio and the period for lateral movement in the x direction were found to be 35.4% and 0.23 s, respectively.

The maximum displacement of the building was found to be 16.9 mm under vertical loads. Displacements observed under fixed and reduced live loads are shown in Figure 17. The walls of the amphitheater and the north façade walls exhibited the highest displacement values.

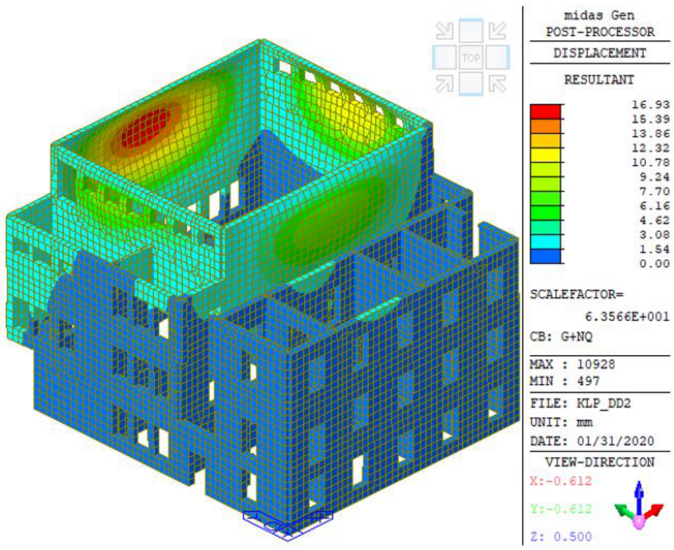

Figure 17. Displacements observed under dead and live loads (G+nQ) (G:Dead load, Q: Live load, n: Live load reduction factor)

The displacements observed under earthquake combined load effects in the positive x direction are shown in Figure 18a.

Under this loading, the wall of the amphitheater on the north façade of the building exhibited the maximum displacement of 93.1 mm. The displacement distribution in the x direction was observed to increase from the lower part of the 1st floor to the top of the building. The displacements observed under combined load effects in the negative x direction are given in Figure 18b.

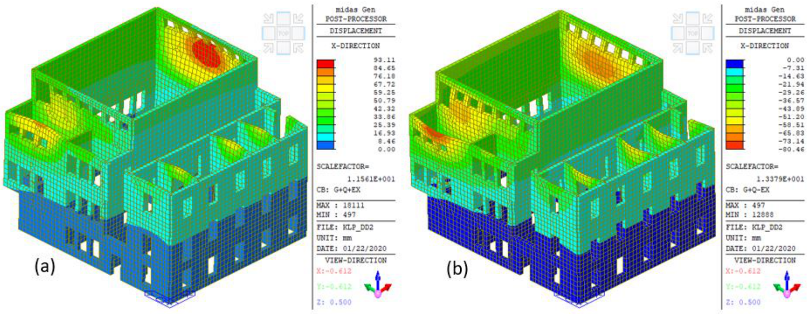

Figure 18. Lateral displacements under earthquake effects in the x direction (a) G+Q+Exp, (b) (G+Q-Exn) (G:dead load, Q: Live load, Exp: Earthquake load in the positive x direction)

Under this loading, a maximum displacement of 80.5 mm was observed on the 2nd floor, exterior wall of stair hall no 202 on the west façade. Additionally, displacement values up to 73 mm were observed on the west façade. The amphitheater walls on the east façade also experienced significant displacements.The displacements observed under earthquake combined load effects in the positive y direction is shown in Figure 19a, a maximum displacement of 147.2 mm was observed in the amphitheater wall on the north façade of the building. Since the first dominant mode of the building was in this direction, the maximum displacement was observed in this direction. The displacements observed under earthquake combined load effects in the negative y direction are shown in Figure 19b. Under this loading, a maximum displacement of 110.9 mm was observed in the amphitheater wall on the north façade of the building. Since the first dominant mode of the building was in this direction, significant displacements were observed in this direction.

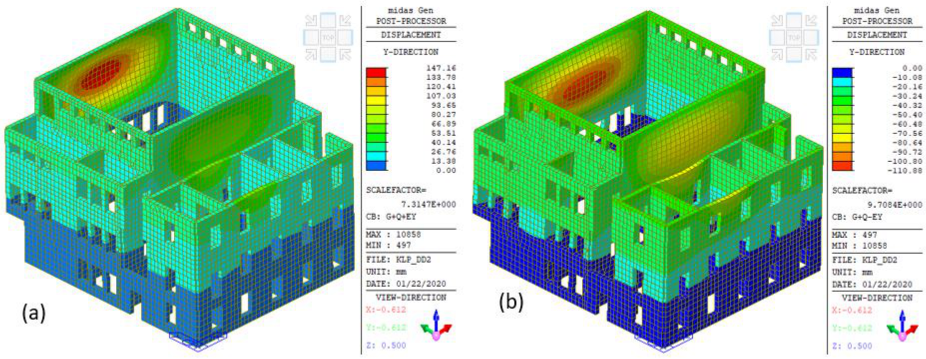

Figure 19. Lateral displacements under earthquake effects in the y direction (a) G+Q+Eyp, (b) (G+Q-Eyn) (Eyp: Earthquake load in the positive y direction, Eyn: Earthquake load in the negative y direction)

Figure 20a displays the stresses induced by the earthquake combined load effects in the positive x direction. These stresses represent axial stresses acting in the x direction, which is perpendicular to the y-z plane. An examination of the observed stresses showed that the maximum axial stress of 4929 kN/m^2 was observed on the exterior wall-wall connections of the north façade. Compression effects on the west and east façade walls resulted in tensile stresses, mostly observed on the north and south façade walls. The stresses observed under earthquake combined load effects in the positive y direction are shown in Figure 20b, representing axial stresses in the y direction perpendicular to the x-z plan. An examination of the observed stresses revealed that the east façade walls, around the window openings of the amphitheater, exhibited the maximum stress of 5690 kN/m^2. The compressive effects on the north and south façade walls resulted in tensile stresses mainly on the west and east façade walls.

Figure 21a shows the axial stresses perpendicular to the x-y plane observed under earthquake combined load effects. The highest compressive and tensile stresses were observed along this axis. An examination of the observed stresses showed that the walls of the amphitheater had the highest stresses, with a maximum stress of 15000 kN/m^2 observed on the west façade walls between the upper window openings.Furthermore, tensile stresses observed under earthquake combined load effects in the positive x direction are shown in Figure 21b. These stresses represent tensile stresses on the y-z plane. An examination of the stresses showed that a maximum stress of 4266 kN/m^2 was observed on the east façade, on the upper window lintels of the amphitheater. These tensile stresses, perpendicular to the earthquake axis were relatively high.

Stresses observed under earthquake combined load effects in the positive x direction are shown in Figure 22a. These stresses

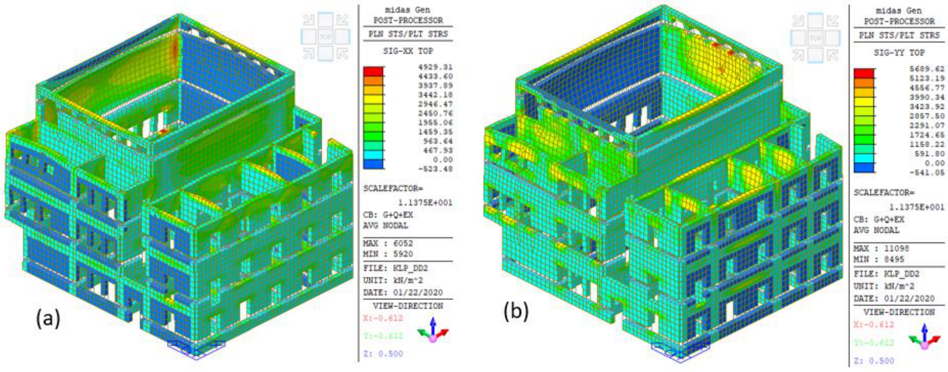

Figure 20. Stresses occurred under G+Q+Exp loading

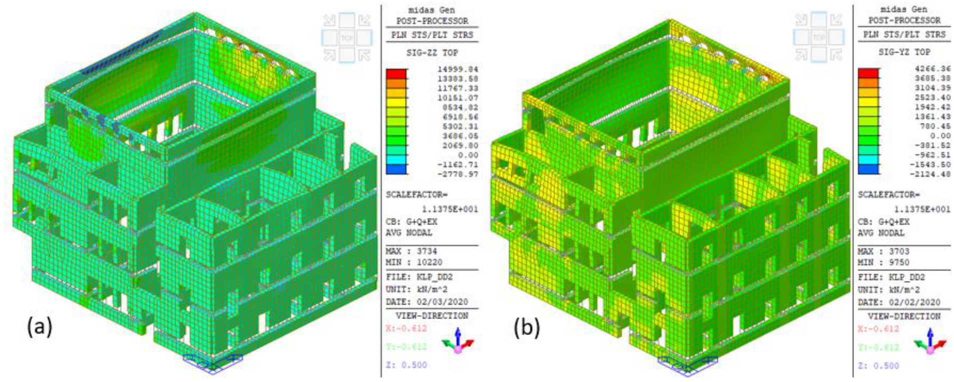

Figure 21. Stresses occurred under (a) G+Q+Exp loading, (b) G+Q+Exp loading

represent shear stresses on the x-z plane. Based on the obtained results, the highest stress value of 2958 kN/m^2 was observed on the exterior walls of the north façade at the 2nd floor. The shear stresses on the window and door lintels of the walls in the x-axis reached a maximum of 1500 kN/m^2. Stresses observed under earthquake combined load effects in the positive y direction are shown in Figure 22b. Upon examining the stresses, it was found that the joints of the exterior walls on the north façade experienced the highest stress, with a maximum value of 5100 kN/m^2. The walls on the y-axis generally experienced compressive stresses, while those on the x-axis experienced tensile stresses.

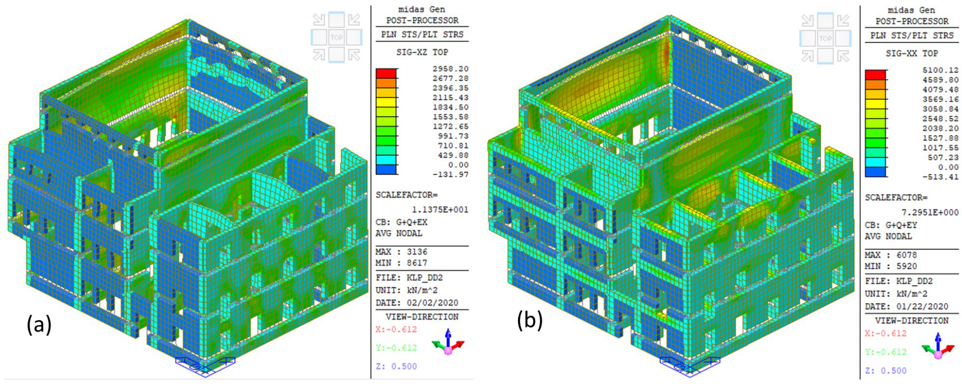

Figure 22. Stresses occurred under (a) G+Q+Exp loading, (b) G+Q+Eyp loading

Figure 23a shows axial stresses observed under earthquake combined load effects in the positive x direction. The analysis revealed that the highest stress was 6608 kN/m^2, which occurred on the east façade between the window openings of the 2nd floor exterior walls. Figure 23b shows axial stresses in the positive y direction. An examination of the stresses showed that a maximum stress of 11318 kN/m^2 was observed on the north façade, on the amphitheater wall and between the window openings and the door lintels of the 2nd floor.

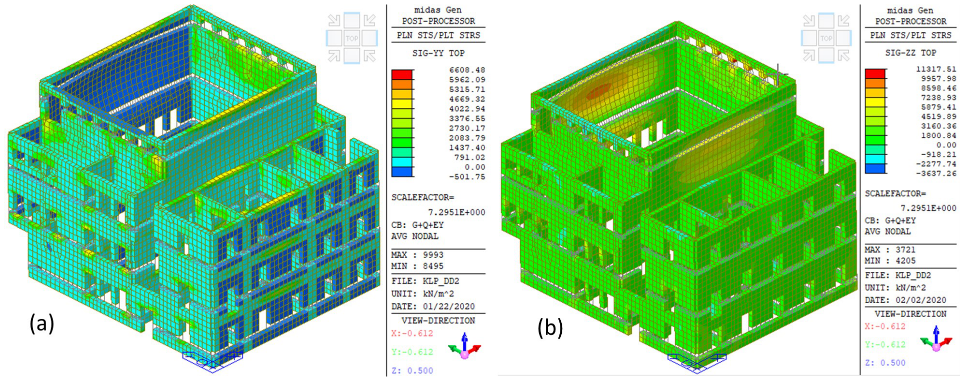

Figure 23. Stresses occurred under (a) G+Q+Exp loading, (b) G+Q+Eyp loading

Shear stresses observed under earthquake combined load effects in the positive x direction are shown in Figure 24a. The obtained results revealed that the upper window lintels of the amphitheater walls on both the east and north façades experienced a maximum stress of a maximum stress of 3671 kN/m^2. Figure 24b shows shear stresses observed in the positive y direction. The obtained stresses revealed that the maximum stress of 3345 kN/m^2 was observed on the upper parts of the exterior walls on the northeast façade. Moreover, shear stresses of 1600 kN/m2 were observed on the window and door lintels of the x-axis walls.

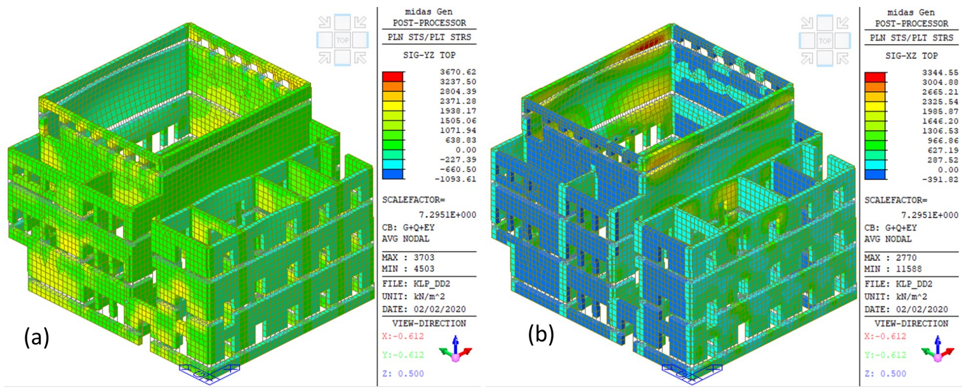

Figure 24. Stresses occurred under (a) G+Q+Exp loading, (b) G+Q+Eyp loading

5. Conclusions and Remarks

In this study, a three-dimension finite element structural model was created and analyzed to assess the seismic safety of a historic masonry building. The results obtained by the seismic analyses indicated that the building does not withstand the shear forces that occurred under both earthquake directions. Therefore, the building was determined to have a poor seismic performance and was at risk of collapse. On the other hand, the building's seismic performance under vertical loads was found to be within acceptable limits. The examination of the building displacements under earthquake effects showed that walls displaced up to 150 mm. Particularly, the highest displacements were observed on the amphitheater walls on the 1st and 2nd floors. Furthermore, the examination of the stresses under earthquake effects showed that stresses reached up to 15000 kN/mm^2. The stresses were observed on the load-bearing members under compressive and tensile conditions due to earthquake loading.

Since there is no regulation specific to historical buildings, seismic analysis and evaluation of such buildings are made according to the principles determined for masonry structures in the Turkish Building Earthquake Regulation.

Performance analysis results were evaluated as follows in accordance with the regulation:

- On the ground floor, the ratio of masonry walls that do not meet the required shear force capacity in the x-direction (100%)

and y-direction (100%) is above the limit value of 40%. As a result, this floor is unable to achieve the desired 'Controlled Damage' performance level and is classified as being in a 'Collapse' state.
- On the ground floor, the ratio of masonry walls that do not meet the required shear force capacity in the x-direction (99.98%) and y-direction (100%) is above the limit value of 40%. As a result, this floor is unable to achieve the desired 'Controlled Damage' performance level and is classified as being in a 'Collapse' state.
- • On the second floor, the ratio of masonry walls that do not meet the required shear force capacity in the x-direction (%99.98) and y-direction (100%) is above the limit value of 40%. As a result, this floor is unable to achieve the desired 'Controlled Damage' performance level and is classified as being in a 'Collapse' state.

Considering the seismic analysis results and the current seismic code requirements, the structure was found to be unable to resist the shear forces generated by earthquakes in both the lateral and vertical directions. Additionally, the building's ability to withstand vertical loads was found to be adequate, but it has poor resistance to horizontal loads.

In conclusion, the building was not seismically safe due to its masonry load-bearing system, structural geometry, material properties, construction conditions, as well as shear capacity. Considering the shear forces, displacements, and stresses observed on the building under earthquake loading, it was recommended that the building should be strengthened as soon as possible before facing a major earthquake.

Current seismic code considers only the shear forces when assessing the seismic performance of masonry buildings. Therefore, many historic masonry structures would be deemed to have poor seismic performance under the current code. However, evaluating historic masonry buildings based solely on their shear capacity and treating them like regular buildings in terms of seismic requirements is not appropriate. To obtain more functional and accurate results while evaluating historic masonry buildings, it is recommended to prepare a separate seismic code specifically for their analysis and interventions.

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