STRUCTURAL CONTROL AND HEALTH MONITORING
Struct. Control Health Monit. 2015; **22**:347–364
Published online 27 June 2014 in Wiley Online Library (wileyonlinelibrary.com). DOI: 10.1002/stc.1677

Seismic evaluation and structural control of the historical Beylerbeyi Palace

Fuat Aras[1,*,†] and Gulay Altay[2]

[1]*Department of Civil Engineering, Istanbul Medeniyet University, Istanbul, Turkey*
[2]*Department of Civil Engineering, Boğaziçi University, Istanbul, Turkey*

SUMMARY

The Beylerbeyi Palace was built between 1861 and 1865 on the Asian shore of Istanbul, and it is within the cultural heritage treasury of the world. Recent earthquakes in the Mediterranean basin have highlighted the particular vulnerability of historic masonry buildings and the need of properly defining retrofitting measures. In this respect, structural behaviour and the earthquake performance of the palace have been investigated. It is seen that the inherent dynamic properties of the palace make it vulnerable to earthquake ground shaking. In order to reduce the affect of ground shaking, the use of base isolation system has been investigated. It is determined that base isolation can be used to reduce the earthquake effects on the palace and ensure the structural performance under maximum considered earthquake. Copyright © 2014 John Wiley & Sons, Ltd.

Received 27 December 2013; Revised 27 May 2014; Accepted 29 May 2014

KEY WORDS: base isolation; historical structures; masonry; retrofit; dynamic analysis

1. INTRODUCTION

Cultural heritage buildings are special structures and must be protected from natural disasters preserving at the same time their authenticity. In the Mediterranean basin, one of the building classes that are consistently exposed to seismic risk is the one constituting the architectural heritage of the region [1]. To minimize further destruction under future seismic activity, it is necessary to reinforce the existing structures that are more vulnerable, but this action always causes some perturbation to the authenticity of these structures as far as the historical evolution of their design is concerned. The main purpose of the engineering analysis of these structures is to obtain the best solution that minimizes the intervention and, consequently, to balance between the need for safety and integrity and the need for preservation of the original structure and tissue [2].

Being aware of the importance of the historical heritage structures and seismic danger of the Mediterranean region, 16 institutions from 12 countries, mostly belonging to the South European and Mediterranean area, have participated in a research project, named as ‘Earthquake Protection of Historical Structures by Reversible Mixed Technologies’ (PROHITECH) [3]. The Beylerbeyi Palace, built between 1861 and 1865 on the Asian shore of the Bosporus, is investigated within PROHITECH. Dynamic identification and numerical analyses have shown that the Beylerbeyi Palace has some inherent dynamic properties, making the palace vulnerable to the earthquake ground shaking.

In this study, base isolation retrofit strategy has been evaluated for the Beylerbeyi Palace. The first historic structure seismically retrofitted with base isolation, the Salt Lake City and County Building, drew attention to the use of isolation for sensitive existing buildings [4]. Later on, seismic isolation

*Correspondence to: Fuat Aras, Istanbul Medeniyet University, D-100 Karayolu, Unalan Yerleşkesi, Uskudar, Istanbul, Turkey.
†E-mail: fuat.aras@medeniyet.edu.tr; fuataras@hotmail.com

was applied to many vulnerable masonry buildings of historic significance, including the Ninth Circuit US Court of Appeals building in San Francisco, Parliament Building and the Assembly Library in Wellington and the Chapel of Rikkyo University in Tokyo [4–6]. Experimental and numerical investigations have also been performed to demonstrate the efficiency of base isolation to retrofit masonry structures [7–10]. Because of the possibility of efficiently improving the seismic capacity of a building with minimal disruption to its architectural features, base isolation system has been recently suggested as an innovative retrofitting strategy, and has been adopted for the seismic upgrading of some major monumental buildings in the USA [7]. In that respect, as an innovative and applicable strategy, a base isolation system, containing high damping rubber bearings (HDRB) is investigated for the rehabilitation of the Beylerbeyi Palace. The structure is modelled with the proposed system and analysed under maximum considered earthquake (MCE). The effects of the base isolation are discussed in detail. Before starting the technical details, it is worth describing architectural and structural characteristics of the palace.

2. ARCHITECTURAL AND STRUCTURAL FEATURES OF THE BEYLERBEYI PALACE

The Beylerbeyi Palace is the largest and most elegant Ottoman palace in Asia. It was designed by Sarkis Balyan during the reign of Sultan Abdulaziz. The sultan attached great importance to this palace and its decorations. The Beylerbeyi Palace illustrates the distinct Western influence on Ottoman architecture. Whilst the exterior side of the Beylerbeyi Palace reflects the Western architectural style, the interior plan complies with traditional Turkish vernacular architecture. The exterior façade is covered by küfeki stone. On the interior wall surface, special stucco plasters with drawings and wood covering with carvings are used [11]. Figure 1 illustrates the palace with exterior appearance and interior spaces.

The palace consists of two main floors and a basement containing kitchens and storage rooms. In the basement floor, storey heights vary between 1.5 and 2.2 m; whereas in regular floors, they vary between 6 and 9 m. The building has a 72 m length along the shore and 48 m in the perpendicular direction. The total height of the structure, excluding the timber roof, can be approximated to 20.60 m. Figure 2 shows the plan layouts of the palace.

The load-bearing system is mainly made of masonry walls and timber slabs. The basement floor of the palace enables to identify the masonry, which is composed of lime mortar, brick and stones. These walls are also forming the foundation system of the palace. The thickness of the walls in the basement floor is generally 1.4 m, whereas it is 80 cm in the first floor and 60 cm in the second floor of the palace. It is determined that the slab of the structure is mainly composed of two types of timber cross-sections: $20 \times 20\,\text{cm}^2$ beams (supporting beam) were located on the masonry walls and $8 \times 40\,\text{cm}^2$ beams

Figure 1. Beylerbeyi Palace ((A) exterior appearance; (B–D) interior spaces).

DOI: 10.1002/stc

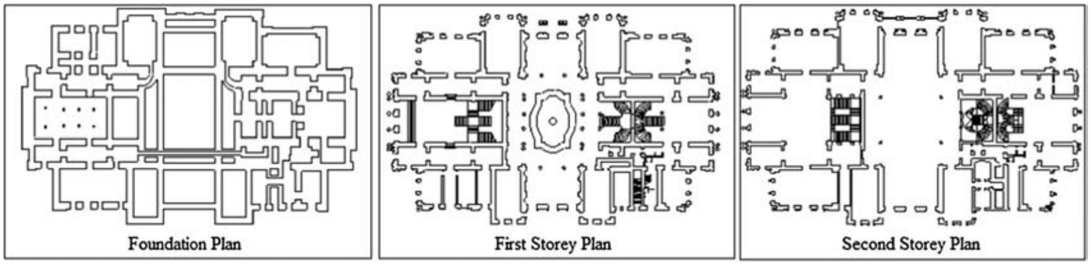


Figure 2. Plan layouts of the Beylerbeyi Palace.

(slab beam) with 40 cm spacing lie between two walls perpendicular to wall direction. Oak was used for the supporting beam, whilst slab beams were produced from fir. Figure 3 shows the structural masonry walls and timber slab system in the foundation and the second floor of the palace.

The Beylerbeyi Palace is used as a museum now. A damage survey carried out in the palace has shown that the palace is presenting the sign of earthquake-oriented damages [12]. For this reason, the palace is investigated within the PROHITECH project [3], and experimental dynamic identification is performed.

3. DYNAMIC PROPERTIES OF THE BEYLERBEYI PALACE

Because of many uncertainties associated with the construction systems, material properties, modelling techniques, analysis methods and soil interactions, the determination of the dynamic properties of a historical structure is indeed a difficult task. Ambient vibration survey (AVS) has recently become the main experimental method for assessing the dynamic behaviour of full-scale structures, and it has generally been preferred for testing historic structures [13].

Dynamic properties of the Beylerbeyi Palace were identified with AVS [14], and the results are used to calibrate the finite element model of the palace. This process is presented in a previous paper in detail [15]. Moreover, the palace adjoins an exterior wall that impairs the symmetrical plans of the structure. However, performed analyses have shown that this wall is not effective in the dynamic properties of the existing palace [16].

For definition of the dynamic characteristics of the Beylerbeyi Palace, ambient vibration measurements were performed at 24 points on the structure, 6 points on four different levels – basement, first storey level, second storey level and roof. For the postprocessing and analysis of the recorded vibrations on all measuring points, ARTEMIS Extractor Pro Software [17] has been used. That software is based on the peak picking technique and frequency domain decomposition and provides good graphical presentation of the obtained data [14]. Figure 4 shows the geometry – built by the measurement points, their respective location on the real structure and the terminology used for the explanation of the mode shapes.

The obtained results are shown in Table I and Figure 5 and showed that the structure can be assumed as fixed supported [14]. The stationary states of the first level in the illustrated mode shapes also support this fact. The obtained mode shapes are showing complex vibration of the structure, although it has a regular and symmetric shape in plan and in elevation. Thereby, the partial movements

Figure 3. Masonry walls and timber slab system in the Beylerbeyi Palace ((A and B) foundation floor; (C) second floor).

DOI: 10.1002/stc

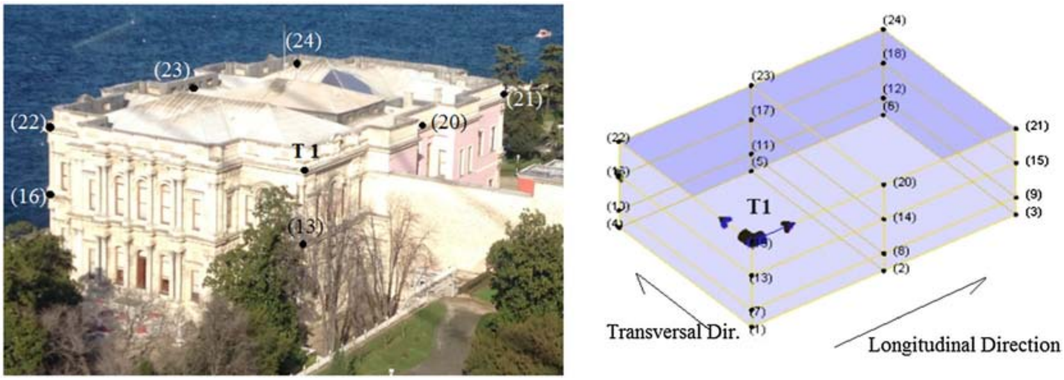
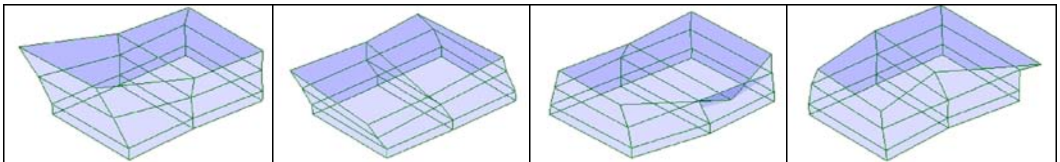


Figure 4. Position of the measuring points.

Table I. Modal properties of the palace, identified by ambient vibration survey.

Mode	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8
Period (s)	0.371	0.278	0.241	0.192	0.126	0.120	0.095	0.069
Damping ratio (%)	2.653	2.6	1.937	2.516	2.542	2.397	1.045	2.549



Mode 1: Movement of the left side along with the TD	Mode 2: Body movement of the structure along with the LD	Mode 3: Movement of the land side-middle part along with the TD	Mode 4: Movement of the left and right side together along with the TD

Figure 5. Obtained mode shapes by ARTEMIS Extractor.

form the mode shapes like in the first and third mode. The complexity seen in the mode shapes is caused by the lack rigid floor diaphragms and nonuniform material properties of the walls. These dynamic properties also make the external wall, seen in Figure 4, as ineffective in the dynamic properties of the palace [16].

The obtained data of AVS are important but can provide more meaningful results if they are used to update a finite element model of the building [18–22]. In that respect, finite element model of the palace has been constructed in SAP2000-V10 computer package [23], and dynamic analysis has been performed. Aiming to fit the modal properties, obtained from numerical analyses to those of the AVS, a tuning procedure has been employed. Presented in another publication [15] in detail, the tuning process has resulted in three different moduli of elasticity for brick masonry in the palace (Figure 6). Although there are always some discrepancies between the real behaviour of structures and numerical analyses, this model is the most appropriate mathematical model representing the dynamic behaviour of the real structure. Earthquake performance of the palace and the effect of retrofit strategies can be examined with this model accurately. Dynamic properties obtained from the numerical analysis of the tuned mathematical model are listed in Table II.

4. EARTHQUAKE HAZARD IN THE BEYLERBEYI REGION

In order to have an appropriate safety assessment for a structure, not only a well-prepared computer model but also the nature of the risk is required. On the basis of recent findings, a fault segmentation model for the Marmara Sea region is developed by Erdik *et al*. [24]. By taking the National Earthquake Hazard Reduction Programme (NEHRP) [25] B/C boundary (characterized with a 30 m average shear

DOI: 10.1002/stc

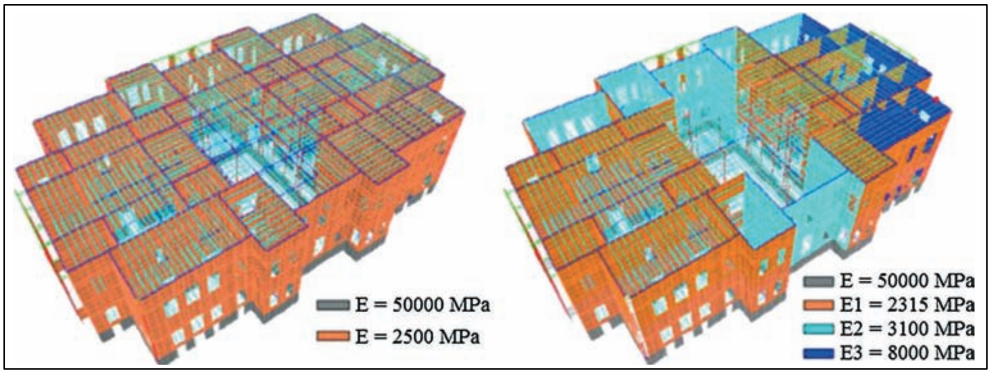


Figure 6. Numerical model of Beylerbeyi Palace before and after the modal tuning.

Table II. Dynamic parameters of the tuned numerical model.

Mode	Period (s)	Mass participation ratio in LD	Mass participation ratio in TD	Total mass participation ratio in LD	Total mass participation ratio in TD
1	0.3706	0.0000	0.1200	0.0000	0.1200
2	0.2887	0.0054	0.0000	0.0054	0.1200
3	0.2809	0.1200	0.0010	0.1254	0.1210
4	0.2701	0.1900	0.0047	0.3154	0.1257
5	0.2597	0.0025	0.0826	0.3179	0.2084
6	0.2533	0.0342	0.0002	0.3521	0.2085
7	0.2309	0.0035	0.0086	0.3556	0.2171
8	0.2304	0.0049	0.1200	0.3605	0.3371
9	0.2294	0.0000	0.0001	0.3605	0.3372
10–58	—	—	—	—	—
59	0.0090	0.1800	0.0258	0.9	0.74
60	0.0087	0.0257	0.1800	0.9257	0.9200

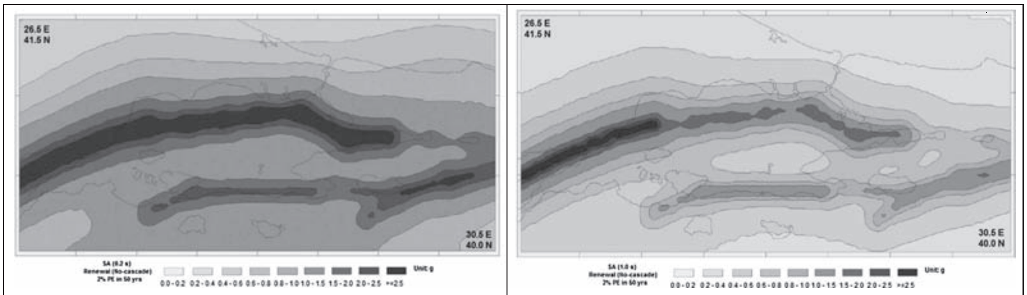
wave propagation velocity of 760 m/s) as the reference ground, hazard maps were obtained for 10% and 2% probabilities of exceedence in 50 years for PGA and spectral acceleration (SA) values at 0.2 and 1 s by Erdik *et al*. [24]. A detailed soil investigation directed by Regional Directorate of National Palaces has identified the soil type that the Beylerbeyi Palace sits on exactly B/C boundary of NEHRP specification [26].

Under these conditions, to determine the earthquake hazard level for the Beylerbeyi Palace, the hazard maps obtained by Erdik *et al*. [24] have been used to determine MCE that has 2% probability of exceedence in 50 years and approximately 2500 years of return period [27]. Figure 7 shows the SA contour maps. The numerical values of SAs are determined according to geometrical coordinates of the Beylerbeyi Palace (private communication with Prof. M. Erdik of Kandilli Observatory and Earthquake Research Institute) as 1.143 g for $T = 0.2$ s and 0.567 g for $T = 1$ s. These values are used to construct the response spectrum of MCE according to NEHRP procedure [27]. The constructed response spectrums for 2%, 5%, 10% and 20% damping ratios are illustrated in Figure 8.

5. STRUCTURAL BEHAVIOUR AND EARTHQUAKE SAFETY OF THE BEYLERBEYI PALACE

Experimental and numerical dynamic analyses of the palace have revealed important characteristics of the structure. These characteristics distinguish the Beylerbeyi Palace from a regular and basic structure. The lack of rigid diaphragm actions on the storey levels and distributed mass through the height of the structure are the two most important structural characteristics that play important role in the behaviour under earthquake loads. The lack of rigid slabs prevents the distribution of lateral forces through the

DOI: 10.1002/stc



SA (T = 0.2 s) contour map for 2% probability of exceedence in 50 years

SA (T = 1.0 s) contour map for 2% probability of exceedence in 50 years

Figure 7. Hazard maps of Marmara region [24].

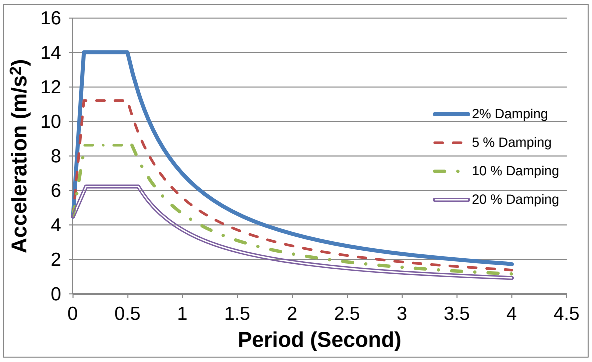


Figure 8. Response spectrum of maximum considered earthquake for different damping ratios.

structural walls. Moreover, out-of-plane failure modes are observed. Instead of lumped mass, distributed mass through the height of the structure cause more complicated dynamic behaviour.

The evaluation of the response spectrum (Figure 8) and the modal parameters (Table II) can give initial precognition about the earthquake safety of the palace. The first 44 modes of 60 modes are on the flat plateau of the response spectrum. This situation makes the Beylerbeyi Palaces affected by the ground shaking severely.

Another important dynamic characteristic, over the earthquake safety of the structures, is the damping ratio. The bigger the damping ratio is, the lesser the earthquake loads acting on the structures (Figure 8). AVS has indicated that the damping ratios are between 1% and 2.7% for the first eight modes of the structure. Generally, the damping ratio of a linear system is assumed as 5%, and the seismic codes give the response spectrum under this assumption [28,29]. In this respect, it can be concluded that; the damping ratio of the Beylerbeyi Palace is such a small value that it increase the horizontal loads compared with a regular system.

The earthquake safety of the Beylerbeyi Palace is assessed by response spectrum analysis (RSA) with the tuned numerical model under characterized MCE. The analyses have been performed for both transversal and longitudinal directions of the structure. In order to include at least 90% of the mass in two orthogonal directions, 60 dynamic modes were accounted (Table II). The outputs of the modal effects were combined by complete quadratic combination method.

The safety of the masonry walls, which are the main structural elements, was concerned. Figures 9–11 show the horizontal stresses (S11), vertical stresses (S22) and shear stresses (S12) under MCE for 2% damping, performed in the longitudinal and transversal directions, respectively. The high stress concentration regions are clearly seen. For horizontal stresses, the upper portion of the structure is under risk. It is interesting to note that the high stress regions have been developed on the walls whose orientations are perpendicular to the earthquake motion. These stresses reach to 7 MPa. Obviously, these S11 stress stem from the out-of-plane movement of the walls and correspond to the bending type deformed shape

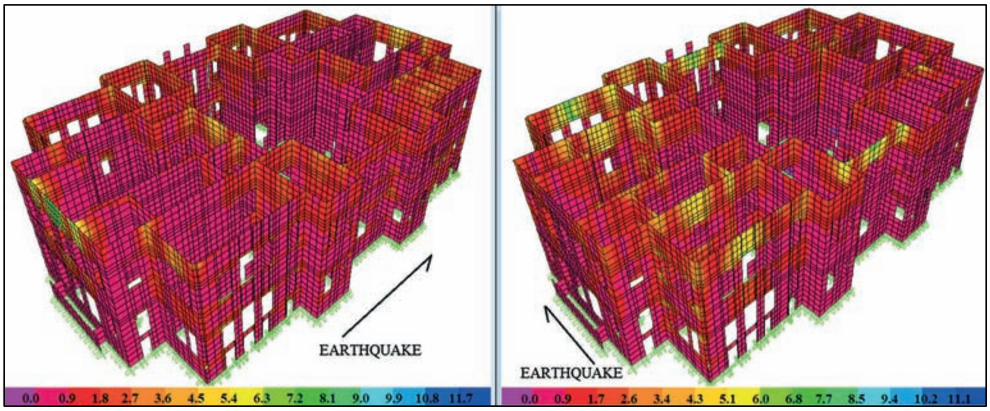


Figure 9. S11 (horizontal) stresses under RSA of MCE for 2% damping.

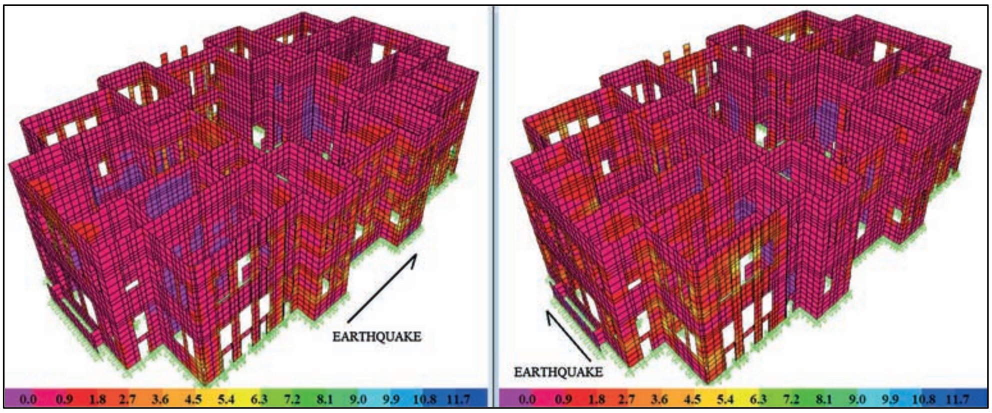


Figure 10. S22 (vertical) stresses under RSA of MCE for 2% damping.

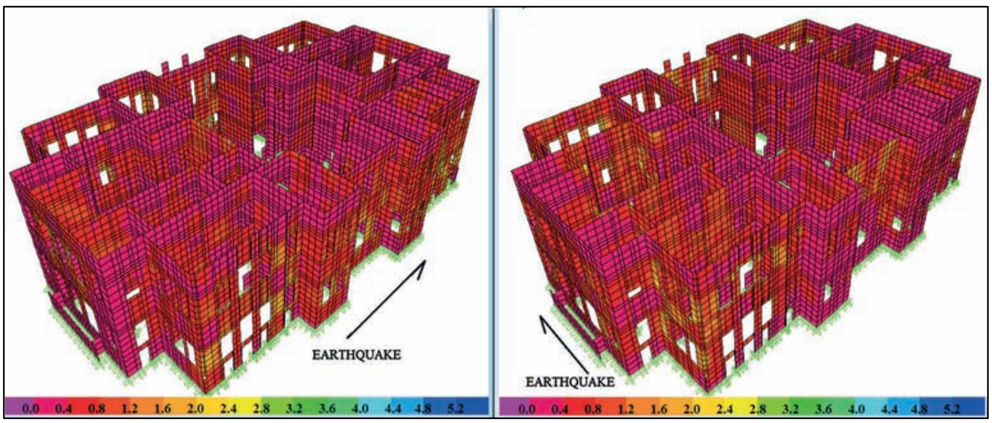


Figure 11. S12 (shear) stresses under RSA of MCE for 2% damping.

(Figure 12). The magnitudes of the vertical stresses are less than those of horizontal stresses, and high stress concentrations are gathered on the lower levels as expected. Additionally, wall segments between two openings are under high stress. S22 stresses are beyond 3.5 MPa. The magnitudes of shear stresses

DOI: 10.1002/stc

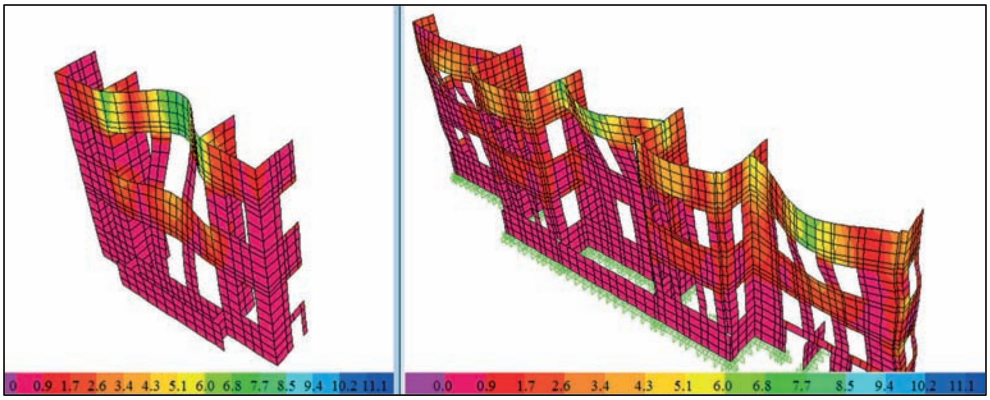


Figure 12. Out-of-plane behaviour and S11 stress concentration.

are less than that of vertical stresses. Generally, its maximum value is about 1.5 MPa, and on the corners of the opening, shear stress concentration is observed.

The creation of tensile and shearing stresses in walls of masonry buildings exposed to strong earthquakes is the primary cause of damage suffered by masonry buildings [30–32]. Although detailed performance evaluation methods, defined in different codes and documents [27,33–36], can be used, basic comparison between developed stresses under given action and material strength parameters can give consistent results related to the safety of the structures. The experiments over the wall and mortar specimens made in the laboratory have given the strength properties for the masonry walls in the palace as 10 MPa for compressive strength and 0.85 MPa for tensile strength [16].

As the result of out-of-plane action of the walls, S11 stresses are well beyond the tension strength of the masonry. This proves that out-of-plane failure occurs under MCE on the higher portion of the walls (Figure 9). Secondly, the compression stresses (S22) are less than the compressive strength of the masonry. Thereby, it can be said that the palace is safe with respect to S22 stresses (Figure 10). Finally, the shear stresses on the masonry walls exceed the tensile strength of the material; however, the shear strength of the masonry depends on the compression stress on the masonry as stated in Equation (1), where τ_{safety} is the expected lateral shear strength of the masonry wall in the system, τ_0 is the masonry cracking shear obtained experimentally, μ is the friction coefficient and σ is the vertical stress on the masonry wall in the system. τ_0 and μ values can be accepted as tensile strength and 0.5, respectively. Evaluation of the stress values showed that most of the walls in the palace are safe under S12 shear stress, but there are many masonry wall segments on which the shear stress exceeds the shear strength of the wall.

$$\tau_{safety} = \tau_0 + \mu\sigma \tag{1}$$

The result of the safety evaluation has shown that the Beylerbeyi Palace is safe under vertical stresses. On the other hand, horizontal and shear stresses exceed the strength parameters of the material. Especially horizontal stresses, because of out-of-plane behaviour of the walls are very high (Figure 12). It can be concluded that the structure is not capable to resist the earthquake ground motion, according to MCE.

6. REHABILITATION OF THE BEYLERBEYI PALACE WITH BASE ISOLATION

Retrofit of a historical monument is an important task and should take into account architectural, historical and structural considerations very carefully. The efficiency of the strategy should clearly be set with respect to application and final performance. The strategy should give the desired performance level. Additionally, historical value of the structure is always of great importance. For this reason, any application should be appropriate and must not disturb the cultural value of the structure.

The use of base isolation strategy is discussed in this study. The isolation approach is based on the response control concept, aiming at controlling and limiting the dynamic effects on the structural elements by means of special devices [37]. The elastomeric bearings are typical isolation devices that

DOI: 10.1002/stc

are composed of alternating layers of steel and hard rubber. This type of bearing is stiff enough to sustain vertical loads yet is sufficiently flexible under lateral forces.

Dynamic modes of the Beylerbeyi Palace showed that the first 44 modal frequencies of 60 modes are on the flat plateau of the response spectrum. For this reason, shifting the fundamental periods is going to result in significant reduction in SA. Secondly, the small damping ratio of the existing palace is another source deficiency. In these respects, HDRB is preferred to isolate the structure. HDRB devices are composed of rubber and steel layers.

6.1. Isolation level and distribution of devices

The insertion of a seismic isolation system in an existing building needs a cut in the structure at floor chosen as isolation level (most often at the level of foundations). This operation is very delicate because it could induce damages in the structural members, especially in a rigid and brittle structure such as the Beylerbeyi Palace.

The isolation devices are planned to be inserted in basement walls of the palace. First of all, these walls also form the foundation of the palace, and their wide thicknesses (1.4 m) are appropriate for the easy installation of the isolators. Secondly, there is no historical fabric on these walls; thereby, the base isolation application does not give any direct damage to the palace's historical authenticity. Thirdly, the analyses of the existing palace have shown that, with their wide dimension, stone masonry basement walls are stiff and safe under earthquake loads. The devices are placed not to transfer the ground shaking to the super structure, but the substructure is still affected by the ground shaking. Verification of the safety of basement walls ensures the safety of the walls under the isolation level. In these respects, basement walls are suitable to instal the isolators with respect to applicability, authenticity and safety concerns.

The installation requires cutting all basement walls and placing two-layered reinforced concrete foundation systems and locating devices between them. The upper layer reinforced concrete system is designed to transmit the structural loads to the isolator uniformly. The role of the bottom layer is to connect the existing foundation and the isolators in order to transfer the effects of the ground shaking to the isolators. In this respect, the isolator should be distributed to the plan of the structure in a way that it does not disturb the load flow and cause torsional behaviour. The selected distribution of the devices is shown in Figure 13A. The total number of bearing is determined as 123.

For the base isolation project, initial point is the determination of the target period of the isolated structure. This is directly related to the amount of decrease in the maximum stress levels. In order to reduce the maximum stress values to acceptable values, the period of the isolated structure should be shifted beyond 2 s. With an additional decrease of damping effect, the stress values on the structure would be acceptable.

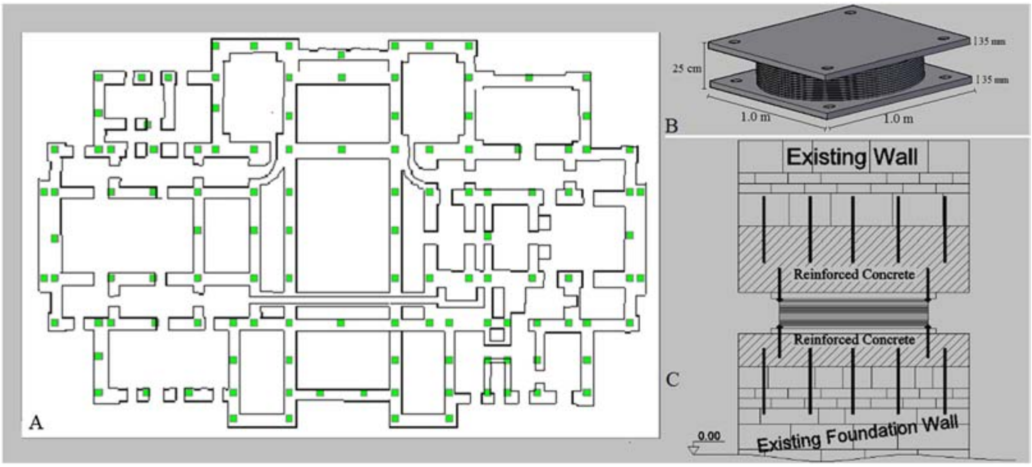


Figure 13. HDRB application in Beylerbeyi Palace (A: Location of the devices on foundation, B: HDRB designed for the structure, C: Application of the devices on the foundation walls).

DOI: 10.1002/stc

Isolator design for the palace has been carried out according to the procedure defined in Yang *et al.* and FEMA 302 [38,39]. Maximum shear deformation (γ_{max}) and effective damping ratio (ζ_{eff}) for the isolator are selected as 150% and 20%, respectively. The International Rubber Hardness Degree 60-type rubber (modulus of elasticity, E = 4.45 MPa) [40] and A275-type steel (yield strength, F_y = 274.4 MPa) are used within the isolators (Appendix A).

The isolator design resulted in the HDRB illustrated in Figure 13B. The diameter of the circular elastomeric portion of the device was determined as 90 cm, and 13 rubber layers of 12 mm thickness were used. The steel plate thickness was determined as 2 mm. Finally, two rectangular steel plates (1 m × 1 m) with a thickness of 3.5 cm have been used on the top and bottom side of the isolator. Figure 13C shows the insertion of the devices to the structure.

6.2. Structural behaviour and earthquake safety of the isolated Beylerbeyi Palace

The efficiency of the base isolation has been investigated by RSA. The numerical model was revised to contain the determined device properties. The HDRB were modelled by beam elements having 25 cm height, geometrical and material properties of devices. The response spectrum, used for the isolated model, has also been revised according to effective damping of isolator (Figure 8).

Table III represents the dynamic characteristics of the isolated Beylerbeyi Palace. The application of isolation has altered the overall behaviour of the structure significantly. Mode shapes of the structure turned to simple rigid body motions, indicating that the isolation task is completely accomplished, and the period of the effective modes are beyond 2 s, indicating a great amount of reduction in SA. The first mode is a torsional mode in which the right side is active in transversal direction, whilst the second mode is the simple movement in longitudinal direction. The third mode is the final mode to complete the overall mass of the structure to 100% mass participation ratio, and it is a torsional mode in which the left side is active in transversal direction. Although the first and the third modes of the isolated structure are formed by the torsional movements in transversal direction, their close periods (2.08 and 2.03 s) eliminate the negative effects. These modes can be altered by changing the design and location of some isolators. However, this may result to an impractical isolation strategy entailing the use of different types of devices positioned at difficult locations.

Figures 14–16 show the S11, S22 and S12 stress values, respectively, for the isolated Beylerbeyi Palace under MCE for two orthogonal directions. The stress values have been reduced significantly. It is clear that the isolated structure is safe under maximum earthquake.

The brittle behaviour of masonry for in-plane and out-of-plane loading is related to limited ductility of the material. For a regular building, in the range of a few millimetres interstorey drift, the masonry walls cracking initiate [37]. In order to see the effect of the isolation, the drift values through the height of the palace are compared for the original and isolated structure under MCE applied on longitudinal and transversal directions. Both in-plane and out-of-plane behaviour have been illustrated via drifts. Four reference lines are taken through the height of the structure, on the midpoint of the exterior walls on the plan. The horizontal displacements, read from the numerical model of the palace, versus the vertical location of the points are sketched on a diagram to see the lateral displacement for the original and base isolated palace through the height of the palace (Figures 17–20). These figures have proven the effect of the base isolation strategy to reduce the relative displacement through the height of the palace. It is clear that the strategy that enables the palace has rigid body motion along with the earthquake directions, thereby absorbing energy and limiting the amount of the force transmitting to the superstructure. As a result, 35 mm relative displacement (between the top and bottom of the structure) for in-plane direction has been reduced to 5 mm (between the top of the structure and the isolator's top anchor). For the out-of-plane behaviour, 78 mm relative displacement (between the top and bottom of the structure) has been

Table III. Dynamic properties of the isolated Beylerbeyi Palace.

Mode	Period (s)	Mass participation ratio in LD	Mass participation ratio in TD	Total mass participation ratio in LD	Total mass participation ratio in TD
1	2.08	0.04	0.49	0.04	0.49
2	2.05	0.92	0.08	0.96	0.57
3	2.03	0.04	0.43	1	1

DOI: 10.1002/stc

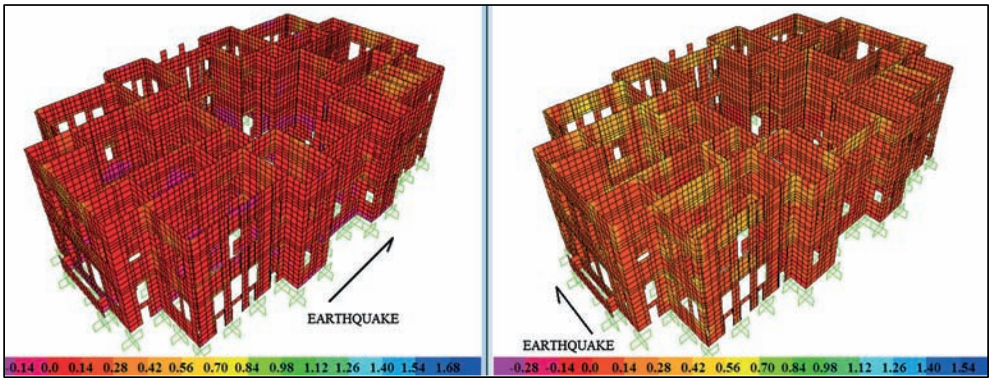


Figure 14. S11 stresses under MCE for isolated Beylerbeyi Palace with HDRB.

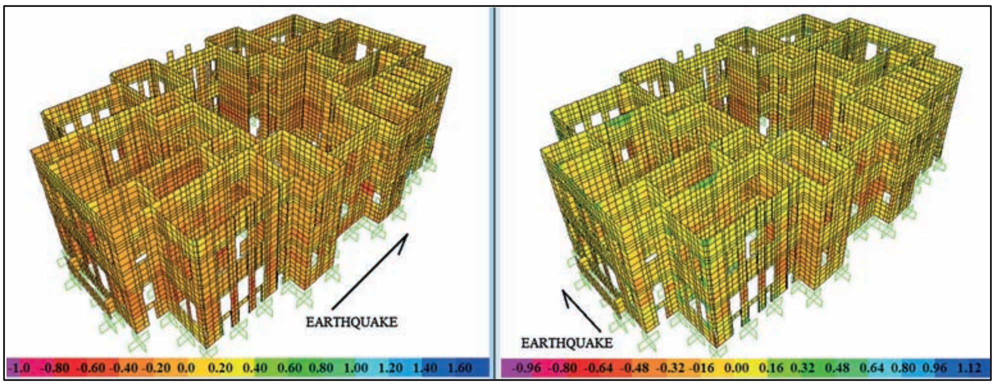


Figure 15. S22 stresses under MCE for isolated Beylerbeyi Palace with HDRB.

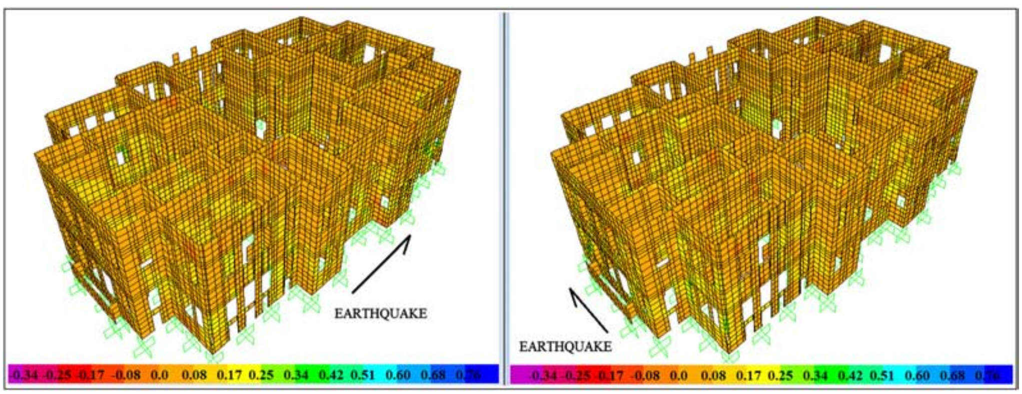


Figure 16. S12 stresses under MCE for isolated Beylerbeyi Palace with HDRB.

reduced to 6 mm (between the top of the structure and the isolator's top anchor). The isolators have experienced around 200 mm displacement both along the longitudinal and transversal directions.

6.3. Architectural and historical aspects of the strategy

The proposed intervention is limited to the foundation level of the building, where a new reinforced concrete beam system should be incorporated together with base isolators. In this way, the facade

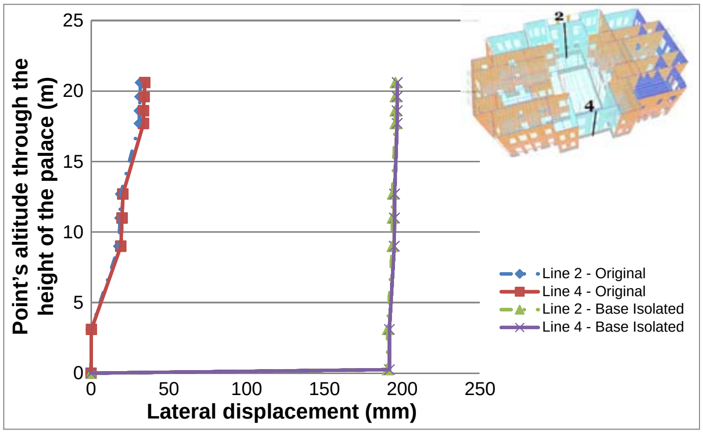


Figure 17. In-plane displacements, through the lines 2 and 4 for the original and base isolated structures under maximum considered earthquake.

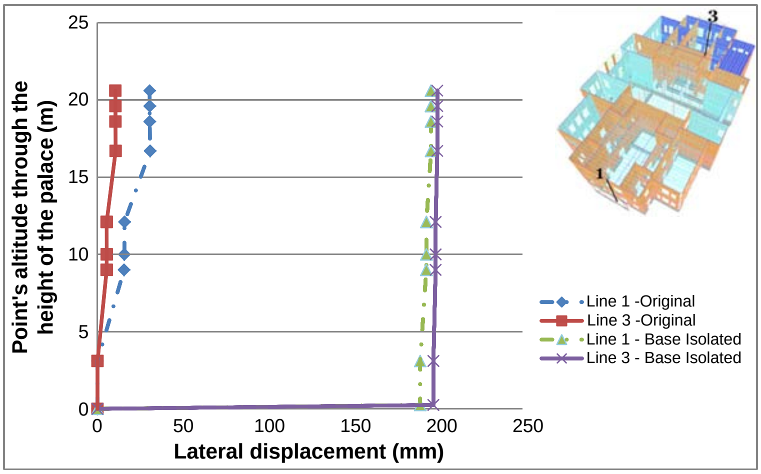


Figure 18. In-plane displacements, through the lines 1 and 3 for the original and base isolated structures under maximum considered earthquake.

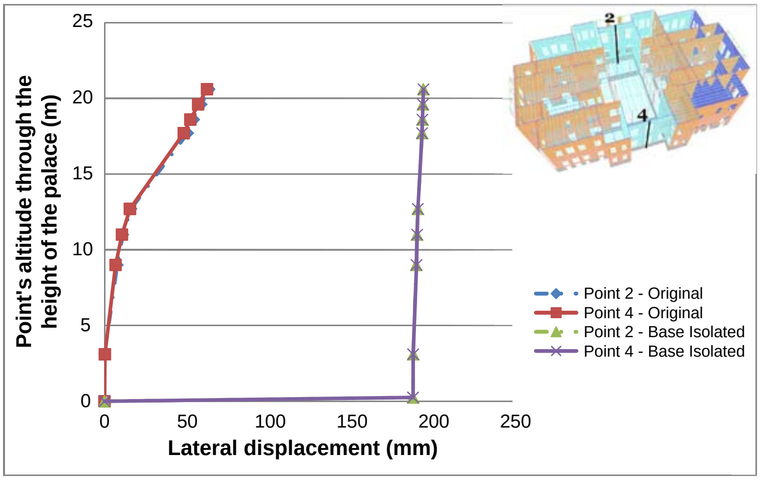


Figure 19. Out-of-plane displacements, through the lines 2 and 4 for the original and base isolated structures under maximum considered earthquake.

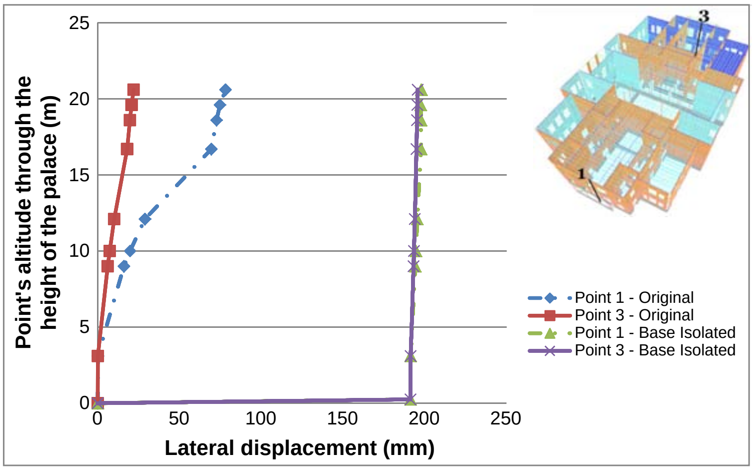


Figure 20. Out-of-plane displacements, through the lines 1 and 3 for the original and base isolated structures under maximum considered earthquake.

and the interior features including frescoes, paintings and other architectural elements are fully preserved. In this respect, seismic isolation is very convenient for aesthetic appearance and architectural features, reflecting the historical importance of the palace.

However, at the level of foundation, an irreversible intervention by adding rigid foundation beams is necessary. Reversibility means the ability to undo the change without harming the original structure. Although base isolation intervention is irreversible, it does not touch the historic fabric of the palace and other historical resources.

Secondly, the intervention technique should also be suitable for the environment of the palace. The Beylerbeyi Palace is actually a palace complex with magnificent gardens, small pavilions and retaining walls. It is obvious that the base isolation strategy has no negative impact on the palace environment. However, there is an adjacent wall joining the Beylerbeyi Palace (Figure 4). The analyses, performed formerly [16], have shown that this wall has no effect on the dynamic behaviour of the existing palace because of the lack of rigid floor diagrams. On the other hand, the proposed base isolation intervention changes the overall dynamic behaviour of the palace, and the structure experiences a rigid body motion under lateral loads. In this respect, in order not to prevent this rigid body motion and cause additional torsion effects for the isolated structure, an additional precaution should be taken to separate the adjacent wall joining to the palace by forming a gap. The width of the gap should be calculated as the sum of the maximum displacement of the isolators and the relative displacement of the masonry wall through the separation line in isolated structure. In that respect, practically 25 cm separation is suitable for the elimination of the effect of adjacent wall into dynamic behaviour of the isolated Beylerbeyi Palace. A separation of the same amount needs to be arranged around the palace, and all lifelines, ducts and the other required links must be connected to the palace via flexible connections.

7. CONCLUSIONS

As a representative of the great historical buildings, the Beylerbeyi Palace was investigated in this study. Shortly, dynamic properties of the palace have shown that lateral ground shaking affects the structure intensively and earthquake performance of the structure is inadequate under MCE. As a result, the retrofit of the palace with base isolation strategy has been investigated, and it is determined that the earthquake safety of the palace can be ensured by the application of the strategy with HDRB devices. The main findings of this study are listed as follows:

- Modal periods of the structure are determined on the flat plateau of the site-specific response spectrum, and the damping ratios for the first eight modes are between 1% and 3%. These dynamic parameters have given the first intuition about the weakness of the palace against the lateral ground shaking.

DOI: 10.1002/stc

- The analyses of the palace under MCE have proven the vulnerability of the structure. Especially, the horizontal stresses (S11) because of out-of-plane behaviour of the walls are seen as the source of weakness.
- In order to heal the dynamic behaviour of the structure and reduce the stress values to the acceptable ranges, base isolation technique with HDRB devices is detailed. A proper device configuration has been designed.
- The analyses of the isolated palace by RSA have shown that the isolated structure is capable to withstand MCE as the result of changed dynamic behaviour and increased damping ratio. The horizontal, vertical and shear stresses are reduced to acceptable ranges.
- Variation of the displacement through the height of the structure has been compared between the original and base isolated structures. It is determined that the base isolation strategy resulted in rigid body motion of the structure under earthquake loads. The average relative displacement between the top and the base (top anchor level of the isolators) of the isolated structure is reduced to one-seventh of that of the original structure. Moreover, the out-of-plane displacements of the walls are reduced to acceptable ranges.
- Lastly, the architectural, aesthetic and historical aspects of the base isolation strategy have been evaluated as acceptable for the palace.

APPENDIX A

DESIGN OF HIGH DAMPING RUBBER BEARINGS FOR BEYLERBEYI PALACE

For the base isolation project, initial point is the determination of the target period of the isolated structure. This is directly related to the amount of decrease in the maximum stress levels. In order to reduce the maximum stress values to acceptable values, the period of the isolated structure should be shifted around 2.5 s. With an additional decrease of damping effect, the stress values on the structure would be acceptable. For the design of the system, the following steps are adopted [38,39]. Each of the parameters is defined at the place where it first appears.

1. Select the material (rubber) properties used in the isolator device and design shear strain and effective damping ratio. Depending on the hardness of the rubber, these values are obtained from the manufacturer's test report. In this study, International Rubber Hardness Degree 60 rubber has been used ($E = 4.45$ MPa, $G = 1.06$ MPa, $\varepsilon_b = 500\%$ and $k = 0.57$) [40]. Design shear strain, γ_{max}, and effective damping ratio, ζ_{eff}, for the bearing are selected as 150% and 20%, respectively.
2. Determine the effective horizontal stiffness, K_{eff}, and design displacement, D_D, of the bearing by Equations (A.1) and (A.2):

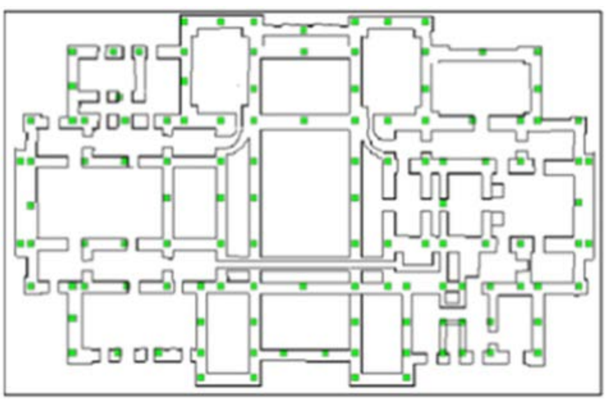


Figure A1. Proposed locations of high damping rubber bearings on foundation.

DOI: 10.1002/stc

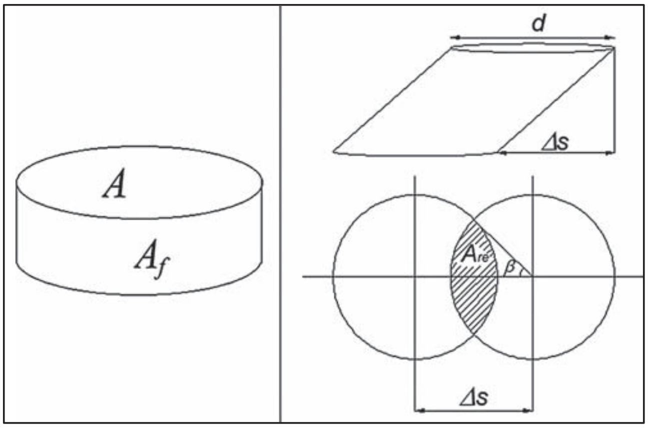


Figure A2. Illustration of load free area, A_f and reduced cross-sectional area of circular bearing.

$$K_{eff} = \frac{P_{DL+LL}}{g}\left(\frac{2\pi}{T_D}\right)^2 \tag{A.1}$$

$$D_D = \frac{g}{4\pi^2}\frac{S_{D1}T_D}{B_D} \tag{A.2}$$

where P_{DL+LL} is the vertical load acting on the isolator, S_{D1} is the design spectral acceleration at 1 s period and B_D is the damping coefficient, given in IBC2000 [25]. According to the number and locations of isolators, the maximum value of P_{DL+LL} was calculated 3200 kN, S_{D1} is 0.44 (modified value of S_1 according to IBC2000) and B_D is specified as 1.5 [25]. K_{eff} and D_D are calculated as 2060 kN/m and 0.183 m, respectively.

3. Calculate the total height of rubber, t_r, in the bearing according to the design displacement D_D and design shear strain γ_{max} by Equation (A.3):

$$t_r = \frac{D_D}{\gamma_{max}} \tag{A.3}$$

where t_r is computed as 0.121 m and selected as 0.15 m.

4. Determine the shape factor S (the ratio of the cross sectional area to circumferential area) by using Equations (A.4) and (A.5):

$$\frac{K_V}{K_H} = \frac{E_c}{G} = \frac{E\left(1+2kS^2\right)}{G} > 400 \tag{A.4}$$

$$S > 10 \tag{A.5}$$

where K_V is the vertical stiffness of the bearing, K_H is the horizontal stiffness of the bearing, E_c is the compression modulus of the rubber-steel composite and k is the modified factor depending on the hardness of the rubber. As the result of Equation (A.4), S is required to be greater than 9.1. In this study, S is selected as 20, and E_c has been calculated as 2034 MPa.

5. Determine the design cross-sectional area of the isolator: It is the minimum of A_0, A_1 and A_2 values specified in Equations (A.6), (A.7) and (A.8):

$$\sigma_c = \frac{P_{DL+LL}}{A_0}. \tag{A.6}$$

Design cross-sectional area A_0 of the bearing based on the allowable vertical stress σ_c under P_{DL+LL} is

DOI: 10.1002/stc

calculated for the allowable value of σ_c as 7.84 MPa. A_0 value has been calculated as 0.408 m^2.

$$6S\frac{P_{DL+LL}}{E_c A_1}\langle\frac{\varepsilon_b}{3} \tag{A.7}$$

Design cross-sectional area A_1 of the bearing is intended to satisfy the shear strain requirement under vertical load P_{DL+LL}. A_1 value has been calculated as 0.113 m^2.

$$A_2 = A_{re} = \frac{d^2}{4}(\beta - \sin\beta) \tag{A.8}$$

$$\beta = 2\cos^{-1}\left(\frac{\Delta S}{d}\right) \tag{A.9}$$

$$A_{sf} = \frac{K_{eff}{*}t_r}{G} = \frac{\pi{*}d^2}{4} \tag{A.10}$$

Design cross-sectional area A_2 is obtained as the reduced cross-sectional area A_{re} (Figure A2). In the calculation, Equations (A.9) and (A.10) are used to determine the β and d, respectively. Δs is the design displacement of the bearing. A_{sf} and respective diameter have been computed as 0.297 m^2 and 0.62 m, respectively. β is determined as 2.127 rad. Finally, A_2 is computed as 0.191 m^2.As the minimum of 0.408, 0.113 and 0.191 m^2, design cross-sectional area of the isolator is selected as 0.408 m^2.

6. Determine the diameter of isolator as it can withstand all possible failure mechanism. In this step, the critical parameter is the reduced area A_{re}. In order to prevent the compression failure when the isolator displaced by design amount, the reduced area must be at least 0.408 m^2.

In that respect, the diameter of the rubber in the isolator is chosen as 0.9 m. In this condition, the cross-sectional area of the isolator is computed as 0.636 m^2, whereas the reduced cross sectional area is 0.470 m^2.

7. Calculate the thickness and the number of individual rubber layers: As specified in Equations (A.11) and (A.12), shape factor, S, is used to determine the thickness of individual rubber layer, t and division of total rubber height by t results the number of required layers.

$$S = \frac{\pi{*}d^2/_4}{\pi{*}d{*}t} = \frac{d}{4{*}t} \tag{A.11}$$

$$N = \frac{t_r}{t} \tag{A.12}$$

The thickness of individual rubber layer, t, was calculated as 1.125 cm and selected as 1.2 cm. Number of layer, N, was computed as 13.

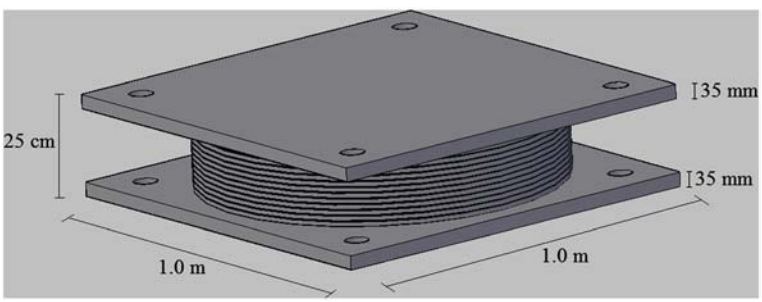


Figure A3. Designed HDRB for Beylerbeyi Palace.

DOI: 10.1002/stc

8. Finally, compute the steel plate thickness t_s by Equation (A.13):

$$t_S = \frac{2(t_i + t_{i+1})^* P_{DL+LL}}{A_{re}^* F_S} \geq 2mm \tag{A.13}$$

where t_i and t_{i+1} are the rubber layer thickness in top and bottom of the steel plate, F_S is 60% of F_y, yield strength of the steel plates (= 274.4 MN/m2). The steel plate thickness is calculated as 1.9 mm and used as 2 mm.

The isolator design has been resulted with the high damping rubber bearings illustrated in Figure A2. Diameter of the circular elastomeric portion of the device is determined as 90 cm and 13 rubber layers of 12 mm thickness have been used. The steel plate thickness was determined as 2 mm. Finally, two rectangular steel plates (1 m × 1 m) with a thickness of 3.5 cm have been used on the top and bottom side of the isolator. Figure A3 shows the designed isolator for the earthquake protection of Beylerbeyi Palace.

ACKNOWLEDGEMENTS

This study has been grown up within PROHITECH (earthquake protection of historical structures by reversible mixed technologies), sixth framework programme project. The authors would like to thank Prof. Dr Federico M. Mazzolani, Prof. Dr Lidija Krstevska and Prof. Dr Lubjimor Tashkov. The assistance, given by Prof. Dr Mustafa Erdik, Assoc. Prof. Dr Waiel Mowrtage, Dr Mine Demircioğlu and Dr Karin Şeşetyan of the Kandilli Observatory and Earthquake Research Institute, is also greatly appreciated.

REFERENCES

1. D'Ayala D, Ansal A. Non linear push over assessment of heritage buildings in Istanbul to define seismic risk. *Bulletin of Earthquake Engineering* 2012; **10**:285–306.
2. Oliveira CS. Seismic vulnerability of historical constructions: a contribution. *Bulletin of Earthquake Engineering* 2003; **1**:37–82.
3. Prohitech. Earthquake protection of historical structures by reversible mixed technologies. Sixth framework programme priority FP6-2002-INCO-MPC-1. Specific Targeted Research or Innovation Project, Abridged version of Annex-I, 2004.
4. Mokha BAS, Amin N, Constantinou MC, Zyas V. Seismic isolation retrofit of large historic building. *Journal of Structural Engineering* 1996; **122**(3):298–308.
5. Robinson WH. Passive control of structures: the New Zealand experience. *ISET Journal of Earthquake Technology* 1998; **35**(4):63–75.
6. Seki M, Miyazaki M, Tsuneki Y, Kataoka K. A masonry school building retrofitted by base isolation technology. *12th World Conference on Earthquake Engineering*, Auckland, New Zealand, 2000.
7. De Luca A, Mele E, Molina J, Verzeletti G, Pinto AV. Base isolation for retrofitting historic buildings: evaluation of seismic performance through experimental investigation. *Earthquake Engineering & Structural Dynamics* 2001; **30**:1125–1145.
8. Tomazevic M, Klemenc I, Weiss P. Seismic upgrading of old masonry buildings by seismic isolation and CFRP laminates: a shaking-table study of reduced scale models. *Bulletin of Earthquake Engineering* 2009; **7**:293–321.
9. El-Attar AG, Saleh AM, Zaghw AH. Conservation of a slender historical Mamluk-style minaret by passive control techniques. *Structural Control and Health Monitoring* 2005; **12**(2):157–177.
10. Syrmakezis CA. Seismic protection of historical structures and monuments. *Structural Control and Health Monitoring* 2006; **13**:958–979.
11. Yücel I. Beylerbeyi Palace, TBMM. Department of National Palaces Publications, Istanbul, 1993.
12. Aras F. Damage assessment and mortar identification in Beylerbeyi Palace. *Gazi University Journal of Science* 2010; **23**(2):211–226.
13. Gentile G, Saisi A. Ambient vibration testing of historic masonry towers for structural identification and damage assessment. *Construction and Building Materials* 2007; **21**:131–1321.
14. Krstevska L, Taskov L. Ambient vibration measurements of Beylerbeyi Palace. IZIIS report no:2006–030, Skopje, 2006.
15. Aras F, Krstevska L, Altay G, Taskov L. Experimental and numerical modal analyses of a historical masonry palace. *Construction and Building Materials* 2011; **25**:81–91.
16. Aras F. Earthquake protection of Beylerbeyi Palace by reversible mixed technologies. *Ph.D. Thesis*, Boğaziçi University, Istanbul, 2007.
17. ARTEMIS. Extractor Pro software, structural vibration solutions ApS. NOVI Science Park, Niels Jernes Vej 10, DK 9220 Aalborg East, Denmark, 2004.
18. Ahmadian H, Gladwell GM, Ismail F. Finite element model identification using modal data. *Journal of Sound and Vibration* 1994; **172**(5):657–669.
19. Mottershead JE, Friswell MI. Model updating in structural dynamics: a survey. *Journal of Sound and Vibration* 1993; **167** (2):347–375.
20. Capecchi D, Vestroni F. Monitoring of structural systems by using frequency data. *Earthquake Engineering & Structural Dynamics* 1999; **28**:447–461.

21. De Sortis A, Antonacci E, Vestroni F. Dynamic identification of a masonry building using forced vibration tests. *Engineering Structures* 2005; **27**:155–165.
22. Chrysostomou CZ, Demetriou T, Stassis A. System identification of an ancient aqueduct. *Smart Structures and Systems* 2008; **4**(2):183–194.
23. SAP2000. Structural analysis program-integrated finite element analysis and design of structures. Analysis Reference, Berkeley, California, 2006.
24. Erdik M, Demircioglu M, Sesetyan K, Durukal E, Siyahi B. Earthquake hazard in Marmara Region, Turkey. *Soil Dynamics and Earthquake Engineering* 2004; **24**:605–631.
25. IBC2000. International building code. International Code Council, Washington, 2000.
26. Eren RH, Oktay FY, Ilkisik OM. Beylerbeyi Sarayi Jeolojik/Jeoteknik Etüt Raporu. TBMM of National Palaces, Report no:05–528, Istanbul, 2005.
27. FEMA-356. Pre-standard and commentary for the seismic rehabilitation of buildings. American Society of Civil Engineers (ASCE), Reston, 2000.
28. Chopra A. Dynamics of Structures. Prentice Hall: Upper Saddle River, NJ, 2001.
29. Hart GC, Wong K. Structural Dynamics for Structural Engineers. John Wiley & Sons Inc.: New York, 1999.
30. Hancılar U, Çaktı E, Erdik M. Earthquake performance assessment and rehabilitation of two historical unreinforced masonry buildings. *Bulletin of Earthquake Engineering* 2012; **10**:307–330.
31. Tomazevic M. Earthquake Resistant Design of Masonry Buildings. Imperial College Press: London, 1999.
32. UNIDO/UNDP. Construction under seismic condition in the Balkan region: repair and strengthening of cultural monuments, Vol. **6**. United Nations Industrial Development Organization, executing agency for the United Nations Development Programme: Vienna, 1984.
33. ASCE/SEI-4106. Seismic Rehabilitation of Existing Buildings. American Society of Civil Engineers: Reston, 2007.
34. Eurocode 6. Design of masonry structures – part 1–1: general rules for buildings: rules for reinforced and unreinforced masonry. European Committee for Standardization: Brussels, 1996.
35. Eurocode 8. Design provisions for earthquake resistance of structures, part 1–4: general rules: strengthening and repair of buildings. ENV 1998-1-4. European committee for standardization: Brussels, 1998.
36. TEC. Turkish Earthquake Code (specification for buildings to be built in seismic zones). Turkish ministry of public works and settlement, Ankara, 2007.
37. Beg D, Skuber P, Pavlovcic L. Set-up of advanced reversible mixed technologies for seismic protection. PROHITECH. Work Package 6 Final Report, Ljubljana, 2006.
38. Yang YB, Chang KC, Yau JD. Base isolation. In Earthquake Engineering Handbook, Chapter 17. CRC Press: Boca Raton, Florida, 2002.
39. FEMA-302. NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures. Building Seismic Safety Council: Washington DC, 1997.
40. Bridgestone Corporation. Multi Rubber Bearings. International Industrial Products Department: Tokyo, 1990.

DOI: 10.1002/stc